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Innovations for Navigation Projects Research Program

Proposed Design Criteria on Thin-Wall Precast Panels for Hydraulic Concrete Structures

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ABSTRACT: This report addresses many of the design issues that designers will face with respect to thin-wall precast concrete panels as the U.S. Army Corps of Engineers continues with its innovative approach to construction of navigation structures using lift-in, float-in, and in-the-wet construction methods.

The report focuses on the issues of concrete panel design that relate to the special considerations associated with the innovative methods currently being used by the Corps. This report is intended to supplement guidance criteria currently available in Engineer Manuals and Engineer Technical Letters, as well as the industry standards such as the American Concrete Institute's building code and the American Association of State Highway and Transportation Officials' Bridge Code.

Special consideration is given to the applicability of thin-wall precast panels, the materials used in their construction, and constructibility issues associated with them. Other topics that are addressed include loads to consider during precast operations, considerations with regard to connection of precast items, issues associated with in-fill concrete, and requirements and guidelines regarding construction details. Several chapters of the report are devoted to issues that are directly related to design, such as serviceability requirements, strength requirements, flexural design, shear and torsion design, and fatigue strength design. Finally, information is presented on design with respect to composite construction and loading combinations that should be considered for thin-wall panels.

Contents

Preface

The research described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The study was conducted under Work Unit (WU) 33237, "Thin-Wall Concrete Panels." Principal Investigator of this work unit is Dr. Stanley C. Woodson of the U.S. Army Engineer Research and Development Center (ERDC) Geotechnical and Structures Laboratory (GSL).

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At the time of publication of this report, COL James R. Rowan, EN, was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

1 Introduction

As the U.S. Army Corps of Engineers has moved to new and innovative methods for constructing navigation projects, the use of precast elements has become much more prevalent. While the Corps has used precast elements for appurtenant items on projects, these elements have not been used for constructing the primary structural components, such as dam and lock monoliths, until recently.

Several projects are implementing precast concrete (P/C) elements in a variety of ways. Braddock Dam on the Monongahela River is being constructed using two large precast units as the overflow sections and lower portion of tainter gate piers. (Lessons learned on the Braddock Project are summarized herein, and four design examples from the project are examined.) Olmsted Lock on the Ohio River will have floating guide walls that will utilize prestressed P/C, while the Olmsted Dam will be constructed using precast segments that will be lifted in place to form the overflow section of the dam and the lower portions of the tainter gate piers.

Precast concrete will also be used on several projects in southern Louisiana. At the East of Harvey Canal Project, a float-in, sector floodgate monolith will be made of P/C, as will sector gate monoliths for the Houma Navigation Canal Lock. Plans for the Inner Harbor Navigation Canal Replacement Lock in New Orleans are to construct the entire lock using precast, float-in units. A float-in cofferdam section is currently being designed for the Kentucky Lock Addition, and this precast float-in unit is also designed to become an integral part of the lock wall. Plans at the Kentucky Lock Addition include a precast floating guide wall similar to the one at Olmsted Lock. The Greenup Lock Extension on the Ohio River will use some P/C towers to provide templates to hang precast panels that will form the lock wall facing. In addition, plans being studied in both the Upper Mississippi River and Illinois River Navigation Study and the Ohio River Main Stem Study include various types of precast elements for lock and dam revisions and replacements.

Because the Corps has used P/C only for appurtenant items in the past, Corps of Engineers criteria for precast, concrete hydraulic structures have not been written. While much of the guidance in current Corps criteria for hydraulic structures can be adapted to precast concrete, there are some items that require further guidance. The Corps' Innovations for Navigation Projects (INP) Research Program work unit "Thin-Wall Concrete Panels" was established to support research for developing this needed guidance. As a product of that work unit, this

report is intended to provide suggested criteria for the design of precast concrete hydraulic structures.

1-1 Objective

This report was written subsequent to distribution of Engineer Circular (EC) 1110-2-6052, "Structural Design of Precast and Prestressed Concrete for Offsite Prefabricated Construction of Hydraulic Structures" (Headquarters, Department of the Army 2001). Much of the information contained in EC 1110-2-6052 is included because it formed the basis for this report. It is intended that this report will extend the criteria provided by the EC and provide a more complete design guide for the design engineer.

While EC 1110-2-6052 provides the basis for this report, additional information has been added through a couple of means. Additional literature review and research has been performed to provide a broader background for the criteria to be used in the design of precast elements. Design of the Olmsted and Braddock Projects has also given designers the opportunity to work within the current criteria and identify shortcomings that need to be addressed. Particular attention has been given to bridging such gaps.

In addition to work under the INP "Thin-Wall Concrete Panels" work unit, the team that developed this report monitored related research being performed for other work units of the INP Program. The earlier INP work unit "Post-tensioning or Prestressing for Hydraulic Structures" had the greatest impact on this research and was the basis for EC 1110-2-6052. Other research performed as part of the "Post-tensioning or Prestressing for Hydraulic Structures" work unit is documented in an earlier ERDC Technical Report (Yao, Gerwick, and Berner 2000).

Another INP work unit, "Low-Density, High-Strength Concrete for Float-in Construction," has some overlap with this research. The focus of that research is on investigating lightweight, high-strength concretes that are suitable for the construction of float-in and lift-in modular units. The use of lightweight concrete as it might affect the final design is discussed in more detail in Chapter 3. Significant findings of this related work unit are summarized in Holm and Bremner (2000).

Another INP work unit, "Modular Connections and Seals for Precast Segments," has emphasis on the connection of various components of float-in and lift-in construction. Thus, it must address issues for these connections with respect to precast concrete. Attaching connection components to precast elements may increase the congestion that occurs within precast panels. Making connection components an integral part of the precast pieces is critical. Further discussion on connections in precast elements, especially the underwater joining of large P/C segments, is contained in Chapter 6.

The "Integrated Design and Analysis System for Navigation Structures" work unit focuses on the development of a computer-aided software package that can provide analytical capabilities that increase both the economy and reliability

of innovative structures. Coordination with this work unit primarily entails ensuring that those team members are aware of the methods and criteria that are suggested in this report.

1-2 Design

Design of the precast elements for the Olmsted and Braddock Projects was performed using existing Corps criteria, supplemented by criteria from the American Concrete Institute (ACI 1999), the American Association of State Highway and Transportation Officials (AASHTO 1998), and the Precast/Prestressed Concrete Institute (PCI 1992). Actual design of the precast sections to be constructed was performed using a combination of design assumptions, hand-calculations, and computer analyses. The designs performed were rational and resulted in structures that were economically feasible and constructible; however, the process used points to a need for developing specific criteria.

With regard to the criteria themselves, the use of criteria from ACI, AASHTO, and PCI is a sound process to employ, although approval from higher authority may be required if the design issue has not been addressed by Corps criteria. These existing standard guidelines generally address precast components for building or bridge structures. It may be inappropriate in some cases to directly use these guidelines for navigation structures without due considerations of special conditions and requirements, such as durability and crack control requirements, and fatigue resistances for permanently floating lock guide walls. For these types of cases, there is a need to first determine whether the criteria in these other codes must be altered, or if they can be used as they are presented. Establishing these issues in official Corps guidance would alleviate the need to obtain approvals for each project. Some discussion regarding design philosophy is contained in Appendix A and may be helpful in making decisions whether or not to use certain codes. Supplemental information presented in other appendixes includes a summary of lessons learned from Braddock Dam construction (Appendix B), a discussion of dimensional tolerances of precast components (Appendix C), example calculations for the design of precast stay-in-place form panels for lock walls (Appendix D), four detailed design examples from recent Corps projects (Appendix E), and a discussion of some alternate design methods (Appendix F).

As noted earlier, design calculations were performed on the Olmsted and Braddock Projects using a combination of design assumptions, hand-calculations, and computer analyses. While the approach was adequate, it may be possible to reduce the extent of design assumptions required, as well as the number of hand-calculations performed. Reducing assumptions during the design process is often valuable with respect to reducing construction cost without affecting the safety of the structure. Use of computer programs can be helpful in this respect. Possible computer applications are discussed below, both for current practice and for possible use in future designs.

1-2.1 Computer applications

In the current state of design practice, the use of computer programs has become more feasible except for the simplest of designs. To make effective use of the designer's time, the design of precast elements for innovative navigation structures should use computer programs to the fullest extent possible. Finite element analysis is often a valuable tool in performing structural design and can range from simple linear elastic, one-dimensional problems to nonlinear three-dimensional (3-D) models. It is often best to use the simplest finite element model that is possible while still being able to accurately capture the behavior of the structural element. For many precast structures to be used on navigation projects, it may be possible to accomplish this through two-dimensional frame elements or 2-D plane stress elements.

On the Braddock Project, a finite element analysis of the overflow section of the dam was performed using grid elements to capture the general behavior of the overflow dam section. This permitted calculation of reinforcing steel in the top and bottom of the precast units for loads being resisted by the entire section. This finite element analysis was supplemented with less complicated finite element modeling and hand-calculations of other portions of the structure, such as ribs in the precast element running longitudinally and transversely along the section.

To perform analyses such as those performed for the Braddock Project, there are several finite element programs that have the necessary capabilities. STAAD-III, GTSTRUDL, SAP2000, and NISA are all PC-based finite element programs that have good preprocessing capabilities along with an element library that is sufficient to solve problems similar to the analysis performed for the Braddock Project. In addition to these finite element programs and others like them, if 2-D frame analysis is being performed, the Computer-Aided Structural Engineering (CASE) program CFRAME (Analysis of Plane Frame Structures) may be useful. For simple problems it should be adequate, is easy to use, and provides the necessary output. For soil-structure interaction under seismic conditions, programs such as FLUSH, SASSI, LPILE, and GROUP are often used.

As problems become more complex, 2-D elements may be required to capture the behavior of a structure. While plane stress or plane strain elements may be needed, output for these types of elements cannot readily be used for performing design. The CASE program CSMT (Calculate Shear, Moment, and Thrust) can be used to take the stresses from a plane stress or plane strain analysis and convert them into shears, moments, and axial forces. This provides an extra level of effort to achieve design results; however, in some cases, it may be necessary to use these types of elements to capture the expected structural behavior.

In addition to the above-mentioned capabilities, work is being undertaken in the INP work unit "Integrated Design and Analysis System for Navigation Structures" to develop a package that can perform 2-D and 3-D finite element analyses and then use these results directly to perform the design. The package will provide a user interface that will permit changes to geometry to be performed quickly and easily. It will also contain a finite element program to analyze the structure. Output from the finite element program will be used to

perform the design based on the stresses produced by the finite element program. This capability will be available for both 2-D and 3-D analyses.

Other possible future applications could be developed under the CASE Program as computer applications are identified to assist in the design of these types of structures. One program that may be possible to update for use on float-in and lift-in P/C units is the CASE program CDWFRM. CDWFRM will perform both static and dynamic analyses of U-frame and W-frame locks. It may be possible to adapt this program for use on P/C units.

1-2.2 Advanced analysis

The analyses described in the above paragraphs are fairly common exercises for structural engineers. Most structural engineers are at least familiar with finite elements and have performed finite element analyses in the past. However, the extent to which the above techniques can be used will sometimes be insufficient to capture all aspects of structural behavior. There will be cases when more advanced analyses will be required, which result in the need to execute finite element programs that are unfamiliar to many engineers.

One example of this case is the performance of a nonlinear, incremental structural analysis (NISA). A NISA is performed using the general-purpose finite element code, ABAQUS (Hibbitt, Karlsson, and Sorenson, Inc. 1999), along with the special-purpose code ANACAP-U (Anatech Research Corporation 1999). ABAQUS is a finite element program that was developed to solve a wide variety of problems. It also will solve very complex problems that most other finite element programs cannot. Performing a NISA also requires familiarity with the constitutive model in ANACAP-U that is used by ABAQUS during the analysis. Proper calibration of the constitutive model is critical to obtaining meaningful results.

Another case where advanced analysis may be required is if the project is located in a high seismic zone. For structures built in such areas, it is sometimes necessary to employ programs that have a wide range of capabilities. A number of programs are available for performing seismic analysis studies. However, these types of programs are often difficult to use, and it is helpful if the individual performing the analysis is familiar with the program.

Another tool to consider is the program IBDAS (COWI Consulting Engineers 2000). IBDAS is a package that allows the user to define the geometry of the structure and, from that geometry, generate a finite element grid. Changes to geometry are made quickly and easily for adjustments that may be necessary during design. The program has capabilities for including the reinforcing in the analysis and can perform a series of code checks as well.

2 Application

2-1 Current Practice

Sophisticated design and innovative construction techniques are becoming common practice within the Corps of Engineers to minimize project costs while maintaining or improving project quality, durability, constructibility, and operability. Use of precast concrete construction can result in lower costs by more effective material usage and reduced on-site labor. The use of P/C can also reduce construction duration by allowing for simultaneous operations, such as the fabrication of large precast units while foundation systems are being installed. New designs are likely to combine precasting of thin-wall units with cast-in-place concrete to provide composite action or to develop continuity. Large precast units are defined as structural elements constructed of two or more precast panels to form float-in or lift-in segments up to several hundred meters (feet) in length.

Precast concrete is concrete that has been cast into the desired shape prior to placement in a structure. P/C components can be designed and used to serve dual functions: forms for cast-in-place concrete and as a durable exterior finish. Precast concrete construction involves concrete forming, placing, finishing, and curing operations away from the project site and erection of the P/C components as part of a completed structure. P/C components can also be used for erosion protection and accessways along riverbanks, coastal shores, and excavated slopes.

P/C construction offers a number of advantages. Precasting operations generally follow an industrial production procedure that takes place at a central precast plant. Thus, high concrete quality can be reliably obtained under the more controlled production environment. Since standard shapes are commonly produced in precasting concrete, the repetitive use of formwork permits speedy production of P/C concrete components at a lower unit cost. These forms and plant finishing procedures provide better surface quality than is usually obtained in field conditions. P/C components may be erected much more rapidly than conventionally cast-in-place components, thereby reducing on-site construction time. P/C components can be designed as an in situ form for underwater construction so that the use of cofferdams may be eliminated or substantially limited. The precasting process is also sufficiently adaptable so that special shapes can be produced economically.

The combination of precasting conventionally reinforced flat panel members joined with second placement concrete (and potentially post-tensioning) has proved economically advantageous for several Corps projects. For example, this type of precast construction has been successfully used for re-facing lock walls, for tainter gate pier construction, and for guide wall construction. (See Section 2-4 for a more extensive list of applications.)

The so-called "in-the-wet" or "offsite prefabrication" construction is an extension of the marine construction methods that have been successfully used before. This innovative method utilizes P/C modules as the in situ form into which tremie concrete or other infill material is placed directly without use of a cofferdam. The precast elements may contain all or much of the primary reinforcement. The tremie concrete is designed to work in composite action with the P/C modules. Numerous investigations and designs have been conducted by several USACE districts and their consultants to evaluate the feasibility of the in-the-wet method at various potential sites of U.S. waterways. These studies have shown that the offsite prefabrication method can provide substantial benefits in cost, construction schedule, risk reduction, facility utilization, river traffic alleviation, and environmental impact. However, there is limited historical evidence to support the benefit of cost savings, and further data are needed to quantify the expected cost reduction.

Precast concrete has its special requirements. First, additional engineering effort is generally required to detail P/C components; develop construction sequences; identify, evaluate, and specify special construction tolerances and specifications; and optimize the design. Second, special labor crews and equipment may be needed to erect the P/C units. Where underwater erection and joining of P/C assemblies are involved, special efforts and techniques are required for positioning, installation, tolerance control, inspection, and quality control (QC).

2-2 Precast Concrete Components: Manufacture and Classification

All P/C components are fabricated offsite as modules and erected on site to make part of a completed structure. Individual P/C concrete members may be prestressed or conventionally reinforced. The manufacture of precast concrete primarily involves three steps: (1) the assembly and installation of reinforcing and prestressing steel; (2) the production and placement of concrete and the subsequent curing; and (3) the lifting, storage and loading-out of the completed component. P/C production is basically an industrial manufacture process. It is appropriate to require more rigorous QC on cleanliness, temperature, and moisture of the aggregates; accuracy of batching and mixing time; and tolerance. Also, the use of an experienced precaster is critical for the manufacture of precast units and should be specified as discussed in Chapter 4, especially for those units with more detail or various embedments. For the erection of precast panels or the fabrication of large, mostly precast segments on site, experienced contractors, labor force, and superintendent are also needed.

P/C concrete normally requires more thorough curing than conventional cast-in-place concrete, because P/C concrete sections are generally thinner and more highly stressed during handling, transportation, and erection. Therefore, adequate means of curing must be specified and enforced. Steam curing at atmospheric pressure is widely employed in P/C manufacture to accelerate the early-strength gain and to permit daily turnover of forms. The adoption of a proper cycle of steam curing and subsequent water curing is essential for good-quality precast concrete. When P/C products are removed from steam curing, the moisture extraction from the concrete is accelerated due to the change in temperature and humidity, so the products must be covered for protection. Similarly, for P/C panels ≥510 mm (20 in.) in thickness, once forms are stripped in cold or windy days, blankets should be draped over them so as to insulate the surface from sudden temperature changes.

External vibration (such as a vibratory table or from vibrators) is useful in eliminating surface defects and for areas congested with rebar or panel embedments. It can also reduce the total time required for complete vibration. However, this method generally cannot extend its effects more than 150 to 200 mm (6 to 8 in.) into the concrete section. For thicker concrete panels, external vibration is not effective to consolidate the inner portion of the concrete. Internal vibration is required for adequate consolidation of concrete in these sections. Vent holes in embedded plates and angles are often used in conjunction with vibration to eliminate voids behind embedments.

In terms of their functionality and structural characteristics, P/C components used in hydraulic structures can generally be classified into two categories: P/C panels and P/C assemblies.

P/C panels are the basic elements that can be used either individually as resurfacing panels or as structural components in a large precast assembly. During the construction stage, individual precast panels are mostly subjected to loads from lifting, handling, wind, and erection bracing. Design considerations for precast panels should follow the relevant guidelines in PCI MNL-120 (PCI 1992) and ACI-ASCE Joint Committee Report 550.

P/C assemblies refer to large prefabricated concrete modules, or segments, for in-the-wet construction of hydraulic structures. These assemblies are typically box-like structures ranging from a dozen feet to several hundred feet in length and width. They are erected at project sites, often under water, as in situ forms into which tremie concrete can be directly placed without use of a cofferdam. For float-in construction, the P/C assemblies must normally have bottom plates to allow flotation, and they contain several compartments to allow sequential ballasting. For lift-in construction, the P/C assemblies normally do not contain any bottom plate, but are fitted with a lifting frame to distribute lifting loads. The P/C units can also be assembled in a dry dock, on a launchway, on a flat-deck barge, in an unused lock, or in a two-level casting basin.

In P/C construction, substantial economy can be achieved through repetition of precasting numerous precast components of the same or similar shape and size. Although prefabrication of P/C assemblies can be carried out in several ways, one frequently used fabrication method is to assemble a large P/C module

by erecting smaller pieces of P/C panels/shells and making closure pours at the junctions of these panels/shells (see also information on connections in Chapter 6, and data on the Braddock Dam Project in Appendix B). Sometimes the individual panels are pretensioned and the assemblies are post-tensioned to meet the strength and serviceability requirements.

Due to their special transportation and erection process, the P/C assemblies used for in-the-wet construction usually experience much more complex loads than those for P/C panels for smaller projects. To design large P/C assemblies, thorough engineering analysis must be conducted for all the critical load cases—during fabrication, outfitting, transportation, erection, and under the service condition. Details of this design and analysis are contained in Chapters 8 through 15.

Another major concern with either float-in or lift-in construction at an existing lock or dam is the impact of any interruption to navigation traffic. Industry costs of up to $1 million per day are possible at busy facilities. Interruptions due to transportation or setdown of the assemblies must be considered, as well as the potential collision with tugs, work barges, mooring elements, or other floating plants. It is important to detail the entire setdown operation to ensure reliable and accurate installation within a favorable weather window. It is also advisable to detail the full setdown operation to minimize the need for the use of divers during construction.

2-3 Erection of Precast Members

For the in-the-wet construction, erection of P/C modules can be carried out by either the float-in method or lift-in method or a combination of both. The float-in construction method entails transportation of prefabricated large concrete modules from their casting yard or outfitting site to the project site through flotation and/or by means of external buoyancy tanks. Once the float-in modules are precisely positioned over the site with a suitable mooring system, or guide piles, or taut lines and winches, they are lowered to the prepared foundation by means of ballasting. Float-in P/C modules usually take the form of floating structures with many compartments for sequential ballasting. This float-in method has been used for years in lowering immersed tube tunnel sections to the bottom by a catamaran crane barge. The prepared foundations may consist of rock-mounted bearing pads, drilled shafts, piers seated into rock, or H-pile or pipe pile foundation systems.

Lift-in construction entails transportation of prefabricated P/C modules from their casting or outfitting yard to project sites by towed barges or floating cranes. The lift-in concrete modules themselves do not float, but may be partially buoyant. Heavy-lift equipment must be used to control positioning of the modules while lowering them onto a prepared foundation to acceptable erection tolerances. Auxiliary guiding systems, such as mooring systems, tensioned guidelines, and guide horns, are often used to assist the positioning. Installation of lift-in P/C segments is largely independent of water level, but is somewhat constrained by riverflow velocity, with a normal upper limit of 2 m/s (6 ft/sec) on

the basis of prior experience. However, units have been installed in current up to 3 m/s (9 ft/sec) where special procedures have been implemented.

Selection of the erection method for the P/C modules is an important design decision. Each erection method has its special implications to project cost, construction schedule, river traffic, towing and mooring system, positioning accuracy, and level of risks during construction. In many ways, the erection method will determine, at least in part, the size and configuration of the P/C modules, foundation treatment, construction sequence, and schedule. In general, a thorough evaluation should be made in the early stage of design to determine the effects of the erection methods, because the erection method and equipment to install precast modules will affect the structural concept and layout, as well as the fabrication of P/C components and construction logistics.

The number of P/C modules and underwater joints between these components has significant effects on the construction cost and schedule. Underwater joining of P/C components is potentially costly and difficult to perform, and the installation of numerous small P/C modules also has adverse implications to the cost and QC (difficult to inspect, resulting in limited quality control). In principle, the use of large prefabricated modules usually provides considerable benefits in costs, project schedule, and construction QC. However, the size of the P/C modules is limited primarily by factors such as the draft requirements, still-water and wave-induced bending moments, and the lift capacity of crane barges that are available or economically obtainable.

For in-the-wet construction, accurately positioning the precast modules under water is one of the critical operations. Underwater positioning of the precast modules requires intensive on-site coordination of several operations, such as surveying; sequential ballasting and/or lift crane maneuvering; operations of guiding devices and hydraulic rams; positioning control with flat jacks, taut lines, and winches; and inspection divers.

Surveys carried out during the erection require a high degree of accuracy for vertical and horizontal alignments of the P/C modules. Engineering specification requires multiple survey systems such as differential global positioning system (DGPS), lasers, and underwater sonic sensors. It is more reliable and expedient for the operators to rely on spotting towers and controlling targets above water than on underwater instrumentation. Therefore, the land-based survey method and GPS should be the primary systems for monitoring the placement, while underwater sonic devices and divers are supplementary methods that cross-check the other survey readings.

2-4 Successful Applications

Many uses for and applications of precast panels, especially thin-wall panels, have been demonstrated for navigation and flood control projects in recent years. These have been constructed both for rehabilitation of older concrete structures and for new facility components. A general list of suitable applications (McDonald 1995) includes the following:

Rehabilitation:

- Resurfacing of lock chamber wall and river wall surfaces.
- Resurfacing of pier vertical surfaces.
- Resurfacing of guide and guard walls.
- Resurfacing of valve wells and pits.
- Rebuilding of lock ladder, mooring hook, and floating mooring bit recesses.

New construction:

- Float-in or lift-in base units for gated dams.
- Float-in or lift-in base units for lock monoliths.
- Precast beams for guide and guard walls.
- Floating guide walls.
- Stay-in-place forms for lock walls (with or without wall armoring).
- Stay-in-place forms for dam piers (with or without nosing steel).
- Baffle blocks or other spillway roughening elements.
- Trunnion girders for gated dams.
- Tailrace panels for dam spillways.
- Scour protection.
- Boat sections for channels.
- Approach walls, both floating and fixed.
- Storm surge barriers.
- Mooring cells.
- Pump stations and concrete barriers for flood control.
- Paving blocks, including those for lock floors.

3 Materials

3-1 Concrete for P/C Units

In general, the standard requirements, recommendations, and restrictions applicable for cast-in-place concrete should also apply to precast concrete. Therefore, materials selection, mixture proportioning, batching, and mixing of P/C should generally conform to relevant provisions of Engineer Manual (EM) 1110-2-2000 and follow the recommendation in the ACI Committee 211 reports, except for the special recommendations discussed below.

Precast and prestressed concrete often has the 28-day compressive strength in the range of 28 to 55 MPa (4,000 to 8,000 psi). Such concrete can be produced with reasonable economy, provided proper care is taken in mixture proportioning and concreting operations. With proper use of water-reducing admixtures and pozzolanic materials, it is realistic and desirable to control the water-cementitious materials ratio (*w/cm*) within the range of 0.35 to 0.43.

High early-strength cements and/or accelerating admixtures are sometimes employed in P/C production to achieve quick form turnover and early prestressing. Such applications may be permitted for production of thin-walled panels. In precasting thick walls or slabs of large concrete masses, high early-strength cements and accelerating admixtures should be prohibited due to the potential problem of thermal cracking and excessive shrinkage. In addition, P/C production often uses steam curing to accelerate the curing time. The impacts of steam curing on thermal behavior should also be evaluated.

Precast concrete usually contains relatively high cementitious materials content. Fine aggregates are typically graded in coarser ranges. The nominal maximum aggregate size should be in the range of 13 to 20 mm (1/2 to 3/4 in.). In highly reinforced components, steel congestion may exist in certain areas, such as at the end blocks of post-tensioning anchorage. In such cases, use of 10-mm (3/8-in.) maximum aggregate size is recommended to facilitate proper placement and consolidation of the concrete.

3-2 Concrete Materials and Mix Design for Infill Placements

The precautions normally applied to concrete materials should be applied to underwater concrete placed by the tremie method. Cement and pozzolans must meet appropriate specifications. Aggregates must be clean, sound, and evaluated for potential harmful chemical reactions. Mixing water must be clean and free of harmful materials. Admixtures must meet appropriate specifications. All materials should be tested to ensure compatibility. Evaluation of thermal behavior due to infill concrete is discussed further in Chapter 7.

a. *Cement and pozzolans.* Selection of the appropriate type of cement and pozzolans must be based on evaluation of service conditions, available aggregates, heat generation, and availability of the materials. The acceptability of a particular pozzolan (workability, water demand, and rate of strength gain) should be verified before final selection. Cementitious materials content must be adequate to produce a flowable and cohesive concrete mix.

b. *Aggregates.* Well-rounded natural aggregates are preferred over crushed angular aggregates because round aggregates generally produce concrete with increased flowability. Aggregates should be well graded. The maximum aggregate size for reinforced concrete is usually in the range of 20 to 25 mm (3/4 to 1 in.) and, for nonreinforced concrete, in the range of 20 to 38 mm (3/4 to 1-1/2 in.). Fine aggregate content should be 42 to 50 percent of the total aggregate weight.

c. *Desired characteristics.* Tremie concrete should not be proportioned on the basis of strength alone. Concrete for tremie placements must flow readily and be cohesive to resist segregation and washout. Standard tests for these characteristics include the slump test, slump flow test, washout test, bleeding test, and time of set test. Appropriate uses and limitations of these tests, and acceptable test results, are discussed in detail in Technical Report INP-SL-1 (Yao, Berner, and Gerwick 1999).

d. *Testing.* The proposed concrete mixtures should be tested using standard American Society for Testing and Materials (ASTM) tests for bleeding, time of set, air content, unit weight, slump loss, compressive strength, and yield to ensure compatibility of components and suitability of the concrete for its intended purpose.

e. *Final selection.* Final selection of a concrete mixture should be based on test placements made under water. Test placements should be examined for concrete surface flatness, amount of laitance present, quality of concrete at the extreme flow distance of the test, and flow around embedded features.

f. *Temperature considerations.* The potential temperature increase should be evaluated using a simple iterative or finite element technique. Assessing temperature rises and gradients may require that a NISA

study be performed. Anticipated thermal gradients should be considered. Based upon the predicted concrete temperatures and gradients, and the nature of the concrete placement, a determination of the seriousness of the prediction may be made. Maximum temperatures and gradients may be reduced by using a lower cement content, replacing cement with a suitable pozzolan and limestone powder, precooling of the aggregates, or lowering the placement temperature.

3-3 Reinforcement

Reinforcing steel for precast components is often prefabricated and preassembled on a template or stand, where the location of every piece is marked and wire ties are used to hold the members in place. Except for welded wire or bar mesh prefabricated in the steel fabricator's plant, tying and clamping of reinforcing bars should generally be used instead of welding. Full penetration butt weld and manual welding in prestressed concrete production should be prohibited. Chair supports and other spacer elements should be capable of supporting reinforcing bar mats or cages without deflections that do not meet required rebar cover.

Nonprestressed reinforcement generally consists of deformed bars or welded wire reinforcement (previously referred to as welded wire fabric). Reinforcing bars should be deformed, except that plain bars may be used for spirals or for dowels at expansion or contraction joints. Reinforcing bars are generally specified to be Grade 60. In some situations, Grade 40 or Grade 70 reinforcement may be specified.

a. *Reinforcing bars*. The most widely used type and grade of bars conform to ASTM A 615 Grade 60 and include bars with sizes from No. 3 through No. 11, No. 14, and No. 18. When welding is required or when more bendability and controlled ductility are required, such as in seismic-resistant design, low-alloy reinforcing bars conforming to ASTM A 706 should be considered. It is recommended to use smaller bars closer together than larger bars farther apart to reduce potential crack width. Reinforcing bars should conform to one of the following ASTM specifications:

- A615, Deformed and plain billet-steel bars for concrete reinforcement.
- A 706, Low-alloy deformed bars for concrete reinforcement.
- A 767, Zinc-coated (galvanized) steel bars for concrete reinforcement.
- A 775, Epoxy-coated reinforcing steel bars.

b. *Welded wire reinforcement (WWR)*. This is a prefabricated reinforcement consisting of cold-drawn wires welded together in square or rectangular grids. Each wire intersection is electrically resistance-welded by a continuous automatic welder. Pressure and heat fuse the intersecting wires into a homogeneous section and fix all wires in their proper position. WWR may consist of plain wires, deformed wires, or a

combination. WWR can also be galvanized or epoxy coated. WWR conforms to one of the following ASTM standard specifications:

- A 185, Steel welded wire fabric, plain, for concrete reinforcement.
- A 497, Steel welded wire fabric, deformed, for concrete reinforcement.
- A 884, Epoxy-coated steel wire and welded wire fabric for reinforcement.

Wire sizes are specified by a letter, W or D, followed by a number indicating the cross-sectional area of the wire in hundredths of a square inch. Plain wire sizes use the letter W; deformed wire sizes use the letter D. Wire sizes from W1.4 to W45 and D2 to D45 may be specified. Wire spacing generally varies from 50 to 300 mm (2 to 12 in.).

c. *Corrosion protection*. When coated reinforcing bars are required as a corrosion protection system, the bars may be either zinc- or epoxy-coated and shall conform to ASTM A 767 or ASTM D 3363 (AASHTO M284), respectively. Epoxy-coated reinforcing bars are generally used in structures that are exposed to a saltwater environment.

Galvanized corrosion protection for structural steel embedments, and anchor bolts fall under ASTM standards other than ASTM A 767. ASTM A 767 includes provisions for chromate treatment of the galvanized reinforcing to prevent a reaction between the bar and fresh portland cement. The reaction produces hydrogen gas that may result in microcracking of the cement paste at the bar locations. The ASTM standards for steel embedments and anchor bolts do not contain provisions for chromate treatment. Specifications for embedments and anchor bolts should reference the appropriate ASTM with a note that chromate treatment in accordance with ASTM A 767 should be provided.

d. *Splicing reinforcement.* The most common method for splicing reinforcing bars is the lap splice. The effect of congestion that the lap splice has on concrete placement should be considered in design. When lap splices are undesirable or impractical, mechanical connections may be used to splice reinforcing bars. Headed reinforcement, hooks, and U-shaped reinforcement can be used in splices to reduce the size/length of the lap splice.

3-4 Embedded Metals

Embedded metals shall be electrochemically compatible with reinforcing and prestressing steel to avoid galvanic corrosion. Aluminum, copper, and stainless steel should not be used as embedments unless positive measures are taken to ensure absolutely no contact between the embedment and reinforcing steel.

Steel embedments with exposed surfaces, such as anchor bolts, have a tendency to corrode, since they become the anode and the reinforcing steel becomes a large cathode to fuel the corrosion potential. Exposed steel anchor or

other embedments should be epoxy coated or separated from reinforcing steel with plastic spacers.

There are other important considerations for both precast and prestressed concrete members having embedded metals. During steam curing of concrete, metal embedments in precast members may expand more rapidly than the surrounding concrete. Any significant thermal change in configuration during steam curing may lead to the local cracking of concrete, which has little tensile strength when steam is first applied. Sponge rubber gaskets can be effectively used to eliminate the problem.

Prestressed concrete components usually experience substantial dimensional changes due to elastic shortening and plastic deformation of concrete. The deformation may lead to cracking at the corners of the embedments. Design considerations should be given to these dimensional changes, and preventive measures taken against stress concentrations at the embedments.

3-5 Lifting Devices, Couplers, and Connection Devices

A lifting device consists of two parts: the anchorage element embedded in the precast concrete and the attachment element, which is attached to the anchorage to fasten the lifting line to the component. To provide adequate strength, the anchorage may bear against the reinforcement. A simple and common device is to embed several loops of prestressing strands in the concrete, leaving the loop exposed for attachment of the crane hook. Selection of the correct lifting devices depends on a number of factors concerned with the type, weight, configuration, thickness, and strength of the precast component.

The location of lifting devices in the components should be carefully considered, taking full account of the special loading that will be imposed on the concrete as a result of tilting, lifting, or moving the component, including an allowance for impact. For example, raising a horizontally cast P/C panel to a vertical position may induce stresses in the concrete that exceed any loading that may be imposed on the panel after it has been installed in a structure.

Selection of the lifting device and its location should be based on the manufacturer's recommendation and an engineering analysis of the proposed installation. The locations and details of lifting and handling devices should be shown on the shop drawings. When requested, the engineering analysis should be reviewed and approved by licensed engineers with sufficient experience with P/C construction.

Lap splices can be used for splicing of reinforcing bars No. 11 or less. Mechanical couplers should be used to splice larger bars. The splicing couplers include threaded bars with couplers, hydraulically forged couplers, and swaged couplers. As in all mechanical items, their use requires proper care in storage, protection in transport and erection, cleanliness, and precision in installation.

Mechanical connections can be categorized as compression-only, tension-only, and tension-compression. Various types of mechanical connections are available that will handle both tension and compression forces. These connectors use a variety of couplers that may be cold swaged, cold extruded, hot forged, grout filled, steel filled, or threaded. In most compression-only mechanical connections, concentric bearing transfers the compressive stress from one bar to the other. The mechanical connection then serves to hold the bars in concentric contact. Tension-only mechanical connections generally use a steel coupling sleeve with a wedge. This is effective only when the reinforcing bar is pulled in tension. In general, a mechanical connection should develop, in tension or compression, at least 125 percent of the specified yield strength of the bars being connected. This is to ensure that yielding of the bars will occur before failure in the mechanical connection. Most mechanical connection devices are proprietary, and further information is available from individual manufacturers. Some have been tested and approved for cyclic fully reversible action and endurance against fatigue. Descriptions of the physical features and installation procedures for selected mechanical splices are given in ACI 439.3R.

3-6 Grout

Grout may be used in a variety of applications in thin-wall construction, including underbase grouting, grouted connections to piles, and grouting between two adjacent P/C segments and between P/C panels and existing structures. Grout may be placed either in the dry or in the wet. Furthermore, either net or sanded grout may be used.

Either site-mixed grout or prepackaged grout may be used to join precast members. Site-mixed grout shall generally conform to the performance requirements specified in ASTM C 1107 in terms of compressive strength, early expansion, and shrinkage. Site-mixed grout shall also meet specific project requirements in consistency, unit weight, and air content through the relevant tests specified in ASTM C 1107.

ASTM Specification C 1107 covers three grades of packaged dry hydraulic-cement grouts (nonshrink) intended for use under applied load. These grouts are composed of hydraulic cement, fine aggregate, and other ingredients and generally only require the addition of mixing water for use. Three grades of grout are classified according to the volume control mechanism exhibited by the grout after being mixed with water:

- Grade A, prehardening volume-adjusting in which expansion occurs before hardening.
- Grade B, post-hardening volume-adjusting in which expansion occurs after the grout hardens.
- Grade C, combination volume-adjusting that uses a combination of expansion before and after hardening.

Performance requirements for compressive strengths and maximum and minimum expansion levels are given in ASTM C 1107. Although these grouts

are termed nonshrink, the intent is to provide a final length that is not shorter than the original length at placement. This is achieved through an expansion mechanism prior to any shrinkage occurring.

Different cementitious materials may be used to produce grout. These include portland cement, shrinkage-compensating cement, expansive portland cement made with special additives, epoxy-cement resins, and magnesium ammonium phosphate cement.

Epoxy-resin grouts can be used between P/C members where increased bonding and tensile capacity is required. When these are used, consideration should be given to the higher coefficient of thermal expansion and the larger creep properties of epoxy grouts.

3-7 Structural Lightweight Concrete

In the prefabrication construction, the weight and configuration of P/C modules have significant implications to their launching, transportation, stability control, positioning, and installation. The size of P/C modules is restricted by many construction constraints, such as draft requirements for float-in construction and the lift capacity of floating cranes available for lift-in construction. Thus, a 10- to 15-percent weight reduction in the modules often leads to significant savings in cost, particularly when buoyancy of the modules is used in construction. In this regard, lightweight P/C modules merit special consideration for navigation structures. In addition to the use of lightweight concrete, heavyweight concrete may periodically have use as ballast, or in paving blocks to prevent uplift.

For structural lightweight concrete, it has been common practice to replace lightweight fine aggregates with normal-weight sand. This is in part because the weight of lightweight concrete is mainly affected by its coarse aggregates. Sand-lightweight concrete has substantial advantages over all-lightweight concrete, including an increase in compressive strength and bond strength, improvement in workability, and considerable reduction in creep and shrinkage of the concrete.

Lightweight aggregates for structural uses are made primarily of expanded clays, shales, or slates. They are light in weight because of the porous, cellular structure of the individual aggregate particles, achieved by gas or stream formation during processing of the aggregates in rotary kilns at high temperature. Concrete made with these lightweight aggregates typically has a unit weight between 14 and 19 kN/m^3 (90 to 120 pcf) compared with about 23 kN/m^3 (145 pcf) for normal-weight concrete. The strength of lightweight concrete can be made comparable to that of natural stone aggregate concrete through proper materials selection, mix proportioning, and control of the *w/cm*. The pozzolanic substances in these aggregates are reactive with the lime in cement paste, forming a stronger bond strength at the aggregate-matrix interface. Furthermore, the porous structure of lightweight aggregates tends to absorb water from the paste around the aggregates, resulting in lower *w/cm* ratios in the transition zone. Microscopic examination of lightweight concrete shows that its transition zone is

much denser, and there is a lack of microcracks that are commonly observed in the transition zone of normal-weight concrete.

The distinctive mechanical and durability characteristics of lightweight concrete are briefly summarized in this section. For more detailed discussions, readers are encouraged to refer to TR ERDC/SL TR-00-3 (Holm and Bremner 2000).

a. *Compressive strength.* For similar mix proportions, the compressive strength of lightweight concrete is inherently lower than that of normal-weight concrete. Lightweight aggregates may have lower strength than the cement paste matrix. However, the specified 28-day design compressive strength of lightweight concrete commonly achieves 35 to 49 MPa (5,000 to 7,000 psi) in major civil works projects. High-strength lightweight concrete can be achieved by a combination of several methods including (1) reduction of water-cementitious materials ratio, (2) use of silica fume or other pozzolanic materials, (3) use of high-quality lightweight aggregates, (4) replacing lightweight fine aggregates with natural sand, (5) high cement content, and (6) use of high-quality aggregate. Lightweight concrete has high average strength-weight ratios in the range of 35 (kilopascals per kilogram/cubic meter), while the average strength-weight ratio of normal-weight concrete is about 27.

b. *Tensile strength.* Both direct tensile strength and modulus of rupture may be related to $\sqrt{f_c'}$. For calculation of deflection, both AASHTO LRFD (Load-and Resistance Factor Design philosophy) Bridge Design Specifications and ACI 318 Code require a reduction factor of 0.85 for sanded-lightweight concrete and 0.75 for all-lightweight concrete. However, the actual relationship is very complex. The safe way to determine the tensile strength is to conduct the test on actual concrete mix (and not to rely solely on the formulas given in the codes).

c. *Shear strength.* The shear strength of lightweight concrete is generally assumed to be lower than that of normal-weight concrete, because aggregate interlock in lightweight concrete is substantially less than that in normal-weight concrete. Both AASHTO LRFD Bridge Design Specifications and ACI 318 Code require a reduction factor of 0.85 for sand-lightweight concrete and 0.75 for all-lightweight concrete.

d. *Bond strength.* For deformed reinforcing bars, the bond strength depends on relative displacement between the bars and concrete. At the working stress range (i.e., small displacement such as 0.25-mm slip), the bond strength is controlled by adhesion and friction. Tests show that lightweight concrete may have higher bond strength than that of normal-weight concrete. As the slip increases, the bond strength depends on the mechanical interlock between the rib and aggregates. Then, normal-weight aggregates have greater stiffness and strength than lightweight aggregates. Both AASHTO LRFD Bridge Design Specifications and ACI 318 Code require a 30-percent increase in steel reinforcement development length for lightweight concrete.

e. *Impact resistance.* Confined lightweight concrete has greater energy absorption under impact loading than normal-weight concrete. If well confined, lightweight concrete can sustain to 6 to 10 percent axial strain without loss of strength. In-service experience showed that lightweight concrete and normal-weight concrete exhibit different behavior under impact loads. Normal-weight concrete showed a small damaged area on the surface, but the internal damage was extensive. Lightweight concrete exhibited a larger area of concrete flaking on the surface, but the internal damage was minimal.

f. *Abrasion resistance.* Abrasion resistance of concrete is highly dependent on the strength and hardness of aggregates. If the surface layer of dense cement paste matrix is removed by abrasion, then some lightweight concrete may rapidly deteriorate due to weak strength of fully exposed lightweight aggregates. Abrasion and erosion properties of lightweight concrete are not fully understood for severe abrasion exposure conditions. For instance, lightweight concrete has shown similar resistance to ice abrasion as normal-weight concrete. Lightweight concrete has shown less resistance from underwater erosion/abrasion from debris and rubble. Until sufficient experimental evidence proves otherwise, lightweight aggregate concrete should generally not be used in applications where the concrete is exposed to rapidly flowing water.

g. *Fatigue strength.* Lightweight concrete generally provides better resistance to cyclic loads than normal-weight concrete, for several reasons. The primary reason is that the stiffness of lightweight aggregates is similar to, or lower than, that of cement paste matrix. Thus, lightweight concrete is more uniformly loaded. On the other hand, the stiffness of normal-weight aggregates is much higher than that of the cement paste matrix. As a result, stress concentration at the interface between matrix and aggregates is higher in normal-weight concrete and fatigue cracking occurs first at the interface–the weakest link in the concrete.

h. *Creep and shrinkage.* In the past, there were reported cases that prestressed lightweight concrete experienced 10 to 20 percent more creep and shrinkage than normal-weight concrete. It has been found that replacing lightweight fine aggregates should substantially reduce creep. Proper lightweight concrete mix design and adequate design allowance should generally eliminate any significant problem associated with creep and shrinkage in association with use of lightweight aggregates.

i. *Permeability.* Permeability of concrete is controlled primarily by its *w/cm* ratio, effectiveness of compaction, and curing, regardless of the type of aggregates used. In general, permeability of lightweight concrete using high-quality aggregates tends to be low, because lightweight concrete does not have the same degree of microcracking and debonding in the transition zone as normal-weight concrete.

j. *Freeze-thaw durability.* Freeze-thaw resistance of lightweight concrete depends on moisture of the concrete and degree of saturation in

aggregates. Some expanded shale or clay aggregates remain 50 to 80 percent unsaturated in concrete. Studies show that the freeze-thaw durability of presoaked lightweight concrete is not significantly different from that of normal-weight concrete, but air-entrained concrete made with lightweight aggregates in the air-dry condition shows higher resistance to freeze-thaw damage than normal-weight concrete. The improvement in freeze-thaw durability of lightweight concrete is primarily due to its low permeability and more air voids in the aggregates to allow frozen water to expand.

k. *Alkali-silica reaction.* Although there is a potential for detrimental alkali-aggregate reaction with some natural lightweight aggregates, manufactured aggregates are pozzolanic and, in fact, prohibit any alkali-silica reaction. There is no known instance of in-service distress due to alkali reaction with lightweight aggregates.

l. *Sulfate attack.* Sulfate attack and delayed ettringite reaction in concrete are basically results of chemical reactions between sulfate ions, lime, and alumina hydrates. In principle, neither lightweight aggregates nor normal-weight aggregates have any effect on these chemical reactions. The best protection against sulfate attack is good-quality concrete with low permeability that prevents penetration of sulfates. Use of proper pozzolanic materials and portland cement containing less than 5 percent C_3A should be very effective in resisting sulfate attack.

m. *Constructibility.* Lightweight concrete has a potential problem in compaction and finishing. Since the aggregates are substantially lighter than cement paste in fresh concrete, aggregates tend to float up under intensive vibration. Overworking the surface while finishing the concrete can cause the same problem by bringing an excessive amount of lighter coarse aggregate up to the surface. To prevent this problem, the concrete must have sufficient cohesion. The amount of vibration compaction and finishing should be controlled to avoid segregation. The pumping of lightweight concrete also presents special constructibility problems. Due to its porous internal structure, most lightweight aggregates can absorb much more water than normal-weight aggregates. When pumped under pressure, the lightweight aggregates will absorb more water from the fresh concrete mix, causing rapid slump loss. A standard practice for pumping concrete requires presaturation of the aggregates. But presaturation of lightweight aggregates can substantially increase susceptibility to freeze-thaw damage. The best solution is to use lightweight aggregates with low absorption.

4 Constructibility Using Precast Panels

4-1 General

This chapter reviews the constructibility of thin-wall precast panels as they are fabricated, transported to the erection site, erected to form larger segments, and then installed into the finished structure. Section 4.6 also provides recommendations and comments on specifications for precast panels. Connections are considered in Chapter 6, and construction details are considered in Chapter 8. Lessons learned from the construction of the new gated dam at the Braddock Locks and Dam in Pittsburgh are provided in Appendix B.

4-2 Panel Fabrication

Panels may be constructed in a prefabrication shop or in a temporary on-site precasting facility. Although many construction considerations are similar for either option, there are some differences. The key to any successful fabrication operation is to provide adequate quality control. Adequate inspection of each step of the fabrication process is necessary to ensure that the panel and its inserts meet all specified material and tolerance requirements.

a. Precast fabrication shop. Special considerations when panels are manufactured in either a full-time precast plant or in a temporary project building or enclosure should include

- Panel size and weight selected to suit the size of the casting bed, building enclosure and crane capacity.
- Concrete mix developed to suit panel thickness, rebar, embedments, and cast-in plates, early strength requirements, and curing methods.
- Fabrication table or casting bed provided to meet finish requirements and tolerances. Ultrasmooth surfaces such as formica-covered tables or beds may be preferred for high-quality surfaces and easy stripping.
- Formwork to meet tolerances, panel stripping, and removal.
- Concrete consolidation to minimize panel honeycombing. When using a fabrication table, it is recommended to use a table vibration system. Pencil vibrators are necessary for tight panel areas.

- Adequate wet or steam curing.

b. *On-site fabrication.* Some contractors may want to fabricate panels in an on-site enclosure or covered casting bed. While the considerations for constructibility are similar to those noted in paragraph *a* above (Precast Fabrication Shop), there are some others unique to this method, as summarized below.

  - Adequate heating of the panels during curing may be a problem during periods of cold weather. A reduction in compressive strength may result from insufficient heating operations.
  - Potentially difficult to meet production schedules.
  - Will often have less experienced staff and require higher levels of QC.

c. *Panel dimensions.* Repetitive panel dimensions and details are important to both improved QC and reduced cost of panels. Some dimensional considerations include

  - Limiting the use of corners less than 90 deg.
  - Limiting the use of L-, C-, or Z-shaped panels. Cracks are likely to develop at interior corners, and special reinforcing steel may be required.
  - Limiting the use of varying depth panels with more difficult formwork and increased consolidation.

d. *Reinforcing bars and inserts.* Constructibility of panel fabrication is highly dependent on the layers and details of reinforcing steel bars and other embedments. If possible, the use of reinforcing bars and other inserts that project beyond the exterior surfaces of the panel should be minimized. Protruding embedments create problems with formwork, concrete consolidation, and panel handling after casting. It is advisable to use flush-mounted inserts for dowel connection during later panel erection.

Reinforcing bar spacing, layers, and concrete cover in panels are also important to providing a constructible panel. Close spacing of bars, bundling of bars, and multiple layers of reinforcing bars add to the difficulty of panel fabrication. If reinforcing bars must be spliced within the panel, it is recommended to stagger the lap splices and to use larger size reinforcing bars. Mechanical couplers and headed reinforcing bars may also help relieve congested panels, but they must also be staggered to avoid congestion.

4-3 Panel Transport

Panels fabricated at a precast plant or at a contractor's permanent or temporary casting facility require transportation to the erection (of large segments) or construction site. Transportation would most likely be by truck or by barge, depending on access to highways and waterways.

Transportation by truck limits the width of panels to 3.65 m (12 ft) to prevent "overwidth" shipping costs. Smaller panels could be shipped vertically and would need to be limited to roughly 2.5 to 3.0 m (8 to 10.0 ft) depending on underbridge clearances. Timber dunnage is required to support and separate adjacent panels when stacked and must be located to suit the structural design of the panels. This will be necessary to eliminate panel cracking during roadway shipping.

Transportation by barge or other waterway vessel is not as restrictive for panel dimensions. However, maximum draft of the waterway must be considered, as well as dimensions of any locks on the trip. Overhead clearance should also be considered for transport below bridges and powerlines.

4-4 Large Segment Erection

The erection of large segments using precast panels and cast-in-place closure pours has come into use for large navigation projects in recent years. Erection of the segments can occur in an existing lock or dry dock, on a flat deck barge, in a graving yard, in a two-level casting basin, or on a launchway or marine railway. Transport is normally by water using float-in or lift-in operations. Segments are sometimes partially or fully post-tensioned after panel erection to improve structural capacity and to reduce segment cracking in key locations.

Erection of panels into large segments requires a suitable foundation system and bracing and shoring to support the heavy panels in proper alignment. Additional shoring may be required for the segment loads during float-in or lift-in operations. Foundation mats or grade beams may be needed for erecting panels in two-level casting basins or in locks or dry docks with earth or uneven floors.

Once the panels have been erected into position and adequately shored, the placement of cast-in-place concrete as closure pours or infill will complete the segment. Intersecting dowel bars or hoops from the panels often makes for a congested placement area inside the forms. Cast-in-place concrete should preferably be flowable and self-compacting, especially for underwater placement. It may be difficult in some cases to provide complete vibration within those tight spaces to ensure no honeycombing and full rebar encasement.

Proper planning of the closure details is necessary during the design and shop-drawing phases. Use of headed rebar may either relieve congestion or make erection of panels less difficult. Smaller aggregate in the concrete mix or concrete admixtures may help relieve placement problems for the closure pours. It is highly recommended to use full-scale mockups of key closure pour areas prior to erection of panels and closure pours on-site. This will help to address any placement problems ahead of real production efforts.

4-5 Panel or Segment Installation

Once the panel or segment (constructed of panels) is completed and sent to the project site, the element is then installed into the new project. Proper support

of the element is required, and proper alignment is necessary to maintain the specified finish tolerances. The use of laser survey equipment and guide frames is often necessary for proper alignment. Landing pads, flat jacks, horn guides, and tapered stabs are common devices for positioning and installing precast panels under water.

a. Panel erection to existing structure. Panels erected into an existing structure often act as stay-in-place forms. As such, they are braced and anchored to the existing structure and are then subjected to infill concrete pressures. Adequate panel reinforcing for the erection and infill loadings and proper shimming for vertical support are also required. Sacrificial bracing members may also be required, especially when an adequate substrate is not available for panel anchorage.

When anchorage is provided into existing concrete, a pullout test program is recommended. A minimum of 10 percent of the anchors should be tested to design capacity, and if anchors are found to fail, then that percentage should be greatly increased.

Proper alignment of water systems and shimming to provide uniform support is important to the installation and backfill operations. Without this support surface, cracking or through-cracking can be expected in the panels during and after concrete infill.

The use of precast panels to resurface lock chamber walls and dam piers has become the state of the practice for rehabilitation since the early 1990s and was documented for the work at Troy Lock by Miles (1993). This repair method has been used extensively by the New York State Thruway Authority for locks and dams on the Erie-Barge Canal and by the U.S. Army Engineer Districts, Rock Island, New York, Pittsburgh, and St. Louis. A design example for this repair method is included in Appendix D.

b. Segment installation. Once panels have been erected into a larger segment, that segment must be transported and set down on a prepared foundation. The two most common methods of transport of the segments are by float in (e.g., Braddock Dam and Inner Harbor Navigation Canal Lock) and lift in (e.g., Olmsted Dam and Greenup Lock). Usually, more than one segment is used to construct a navigation structure, and the multiple sections sometimes require a closure pour and post-tensioning to tie them together.

Float-in construction requires the segment to be designed as a floating vessel, allowing for wave, current, submersion, and impact forces as described in Chapter 5. The use of one or more tugboats is required, along with a multiple-line mooring and positioning system. Other considerations include

- Draft of segment for dead load, bulkheads, and equipment.
- Floating stability of segment.
- Setdown pads for temporary support.

- Underbase grouting and closure systems to contain the tremie concrete.
- Horn guides or other positioning devices.
- Segment infill with tremie or standard placement concrete.
- Position moorings with winches and taut lines.
- Suitable ballast systems and proper ballasting sequences.

Lift-in construction requires the segment to be designed for lifting by large crane(s) or barge-mounted cranes or winches. This method will most likely be transported over water, except for short-distance land transport by cranes. A wide range of special heavy lift-in techniques and equipment are documented in Technical Report ERDC/GSL TR-00-2 (Gerwick and others 2000). Special considerations for the lift-in method include

- Weight and configuration of segment and crane/winch capacity.
- Availability and mobility of lift crane.
- Guiding devices such as guide frames and horn guides.
- Temporary supports such as setdown pads and grade beams.
- Closure systems to contain the tremie concrete, such as enclosure seals and skirts.
- Segment infill with tremie or standard placement concrete.

4-6 Use of Specifications for Improved Constructibility

Part of having a constructible project is to have specifications that ensure a quality product. Since the use of P/C elements on civil works projects has been limited to a few components of navigation structures until the past 5 years, attention to specifications for precast elements has not been given a great deal of attention by the Corps of Engineers.

Several topics (discussed in the remained of this section) can provide the insight needed for developing specifications for certain areas. This information provided is done so with the intent that designers with little experience in P/C will be able to review and develop the specifications with a better understanding of the primary concerns regarding the use of precast concrete. The items that are discussed do not represent a complete listing of considerations for developing specifications for P/C elements. Rather, these items were chosen because related concerns were raised by those involved in some of the Corps' first large, precast navigation structures.

Both the PCI and the ACI have valuable resources, from which much of the information in this section was obtained. Those who are developing specifications for P/C elements are directed to the publications of these two organizations for further, in-depth discussions on various topics related to precast concrete. The more pertinent of these publications are referenced where appropriate in the paragraphs below.

4-6.1 Materials

Many concerns regarding precast materials are similar to, if not the same as, concerns that are present with respect to cast-in-place concrete. Both types of concrete should require submittals with respect to mixture proportions, material characteristics, and properties of freshly mixed concrete. Uniform input for these types of submittals may be advantageous, and ACI 211.5 has examples of forms that can be used for various submittals. Presence of oils, acids, alkalis, salts, organic materials, or excessive amounts of chloride ion in the mixing water should always be avoided, regardless of where casting takes place.

A variety of approaches can be taken to revise the behavior of the concrete so that it is more suitable for a given application. For instance, using more aggregate in the mixture will reduce the shrinkage of the concrete, which can in turn reduce the cracking that occurs, while increasing water content will increase drying shrinkage. Care must also be taken when using aggregates with silica, since the alkali in cements will react with the silica and form an expanding gel in the concrete, causing it to expand and often crack or create other structural problems. A similar behavior can occur if sulfate-bearing waters come into contact with hydrated calcium aluminate. However, use of ASTM C 150 Types II and V portland cement will reduce this problem.

For information beyond the general guide specifications, ACI 301 contains an extensive checklist of optional requirements that may be considered for inclusion on projects that have concrete placement. This would be an excellent document for an engineer to review prior to any project that will use concrete in construction, both precast and cast-in-place.

There are, of course, some items that are more specific to P/C materials than cast-in-place. ACI 357R suggests that portland cements conforming to Types I, II, or III be used in precast construction. This document also notes that, in these types of applications, additional testing may be needed beyond typical concrete testing. These tests might include splitting and flexural tensile tests. ACI 357R also notes that durability can be improved by using hard, dense aggregate.

Requirements should be included for dry, covered storage areas for the storing cement for precasting operations. In addition to this requirement, ACI 533R states that cement should be sampled from each shipment received and kept in a sealed container. For precast concrete, a good quality control program is critical in achieving a quality product. This can be achieved by good record-keeping, which makes the QC process simple and consistent.

4-6.2 Certified precast site

The PCI recommends in the PCI Design Handbook (1992) that, when using precast concrete, a certified precast site should be used. The use of a certified precast site ensures that quality assurance (QA) measures are exercised at the site and are checked on a regular basis. Since contractors will often establish a precast yard at or near the assembly site, the specifications should require that the site become certified by PCI, or the specifications should be written so that the

requirements meet those needed to become certified. The PCI Design Handbook contains suggestions for specifications that outline the requirements a contractor should be capable of meeting.

One of the primary documents that should be referenced if not using a certified precast cite is the publication "Manual for Quality Control for Plants and Production of Precast and Prestressed Concrete Products" (PCI 1999). This document will enable those who write specifications to determine the necessary and important aspects of QA and QC for precast concrete. This document will help ensure that a precast site will provide a quality product even though a certified precast site is not used. However, this can only be achieved through the project specifications, and only if they are clearly written.

In addition to the items described above, ACI 533R also points out the need to ensure that responsibilities for precast items are well defined in contractual agreements. This document includes discussions on how reinforcement should be handled; on ensuring the adequacy of connections, erection sequences, and temporary shoring; and on the importance of clarifying the responsibility for each of these activities. It is suggested that the contractor be required to perform a pre-erection check prior to assembling precast components. It may also be valuable to require the contractor to do full-size mockups of connections to identify any areas in the connection that could result in a problem when trying to erect the actual structure.

ACI 533R also emphasizes the importance of conversations between the engineer and the precaster prior to precasting, as well as during transportation and erection. Elsewhere, it is suggested that a construction and installation manual should be developed by the engineer to describe any critical operations that may take place during construction, transportation, and installation (ACI 357R).

Specified tolerances are an important part of precast construction. Three types of tolerances exist for precast construction: product, erection, and interfacing tolerances (ACI 533R). Care must be taken when writing the specifications for precast tolerances so that tolerances are compatible, that is, that specifications for one of the three types of tolerances do not require one of the other types to be exceeded. For reference, Appendix C provides general guidelines for specifying tolerances for production and erection of large P/C assemblies.

4-6.3 Lightweight aggregates for structural applications

For many applications of precast elements in navigation structures, the P/C must employ the use of lightweight aggregates so that the draft depths of float-in structures are not too deep, or to achieve structures that can more readily be lifted into place. Lightweight aggregates can be used to reduce the unit weight of concrete by as much as 40 percent. However, for the civil works applications considered by the Corps of Engineers, the weight reduction achieved will typically approach 20 percent or less, as discussed in Chapter 3.

One of the primary concerns when dealing with lightweight aggregates is the fact that lightweight aggregates tend to absorb more water than normal-weight aggregates. It is suggested in ACI 211.2 that lightweight aggregates should be damp prior to mixing, which will reduce the slump loss that may be exhibited if these types of aggregate are dry prior to mixing. Holm and Bremner (2000) state that, if aggregates are used with a moisture content equal to at least their 24-hr absorption moisture content, the concrete can be used without significant slump loss if placed using a conveyor or a crane. However, if pumping is to be used, a higher saturation of the aggregate will be needed. While the presaturation of lightweight aggregates is beneficial in maintaining slump, the concrete resulting from saturated lightweight concretes is more susceptible to freeze/thaw damage than if the aggregates were not presaturated.

Another point given by ACI 211.2R regarding lightweight aggregates is that the mixture proportions for concretes using these aggregates are given for the oven-dry condition, although when they are actually mixed in the field, they are typically in the damp or wet condition. This difference must be accounted for when writing the specifications. It has also been noted that better graded aggregates provide a higher strength and lower volume change due to shrinkage than aggregates that are not well graded. To improve the workability of lightweight concrete, it is recommended that it be air entrained.

Some other items to consider when using lightweight aggregate are the use of fly ash and silica fume (Holm and Bremner 2000). Inclusion of fly ash in lightweight mixtures has shown that these mixtures will perform well, providing the needed density and strength. Adding silica fume has been shown to actually improve the structural properties of the concrete, giving it higher compressive, tensile, flexural, and shear strengths.

4-6.4 Forms

Formwork is, of course, an integral part of precast concrete. There are considerations both for temporary forms and for concrete forms that will be permanent. While in general the designer should not be concerned with the actual design of formwork, ACI Special Publication 4 (Hurd 1995) provides both general and basic information concerning forms and numerous design examples of various structures.

Typically the designer should not specify how to design or build formwork to achieve the specified tolerances (ACI 347R). Design of the forms, how to build them, and what materials to use should be left to the contractor. The contractor needs only enough information so that he can adequately build the forms. This may include specifying that adequate slope of forms be achieved to aid in the removal of air bubbles that may be present in the concrete (PCI 1992). Inclusion of criteria for checking deflection may also be warranted.

Formwork for placement of concrete under water is addressed in ACI 347R. While the design methods for designing formwork are the same, some precautions about placing concrete under water in temporary forms may need to be included in the specifications. A buoyant density of concrete can be used in

designing forms; however, care should be taken in using a reduced density since concrete will usually be placed with a tremie tube, and large loads can sometimes be developed due to the head developed in the tremie. It may be wise to include a note concerning this in the specifications, or to require that an individual experienced in this type of work to evaluate the formwork system.

Unlike temporary forms, if P/C is used as forms, the designing engineer should be responsible for the design and must include any external bracing that may be needed as part of the formwork system (ACI 347R). Both the PCI Design Handbook (1992) and ACI 347R contain information regarding the design of formwork that can be used to design for P/C used as forms. Some of this information is presented in Chapter 5 as well.

It is also recommended that any forms that may be used under water be well detailed if divers are being used, so that they can become familiar with the structure prior to beginning their work.

4-6.5 Creep deflection

Although many items are included in the specifications, actual decisions about these often have to be made in the design of the structure. A brief discussion of some of the things that can be done to reduce deflection is presented below.

A variety of things that affect creep are noted in ACI 209R. Since the thickness of a member affects creep, an increase in member thickness will reduce creep. An increase in moist curing will decrease creep, as will values of ambient air humidity above 40 percent. Therefore, specifying placement of concrete during periods of the year when humidity is historically high may be an option, but typically this may also increase the cost of construction. Some of the things that will increase creep, and deflections associated with creep, are higher slumps, higher percentages of fine aggregates, higher cement contents, and higher air content.

ACI 435R has recommendations for reducing deflections, including those associated with creep. One very effective way to reduce creep deflection is to increase compression reinforcement. However, this becomes less effective as members become more shallow. If loading of the element occurs at a young age, then consideration should be given to specifying a mixture that gains strength early to reduce creep deflections.

To reduce the effects that creep deflection may have on a structure, the PCI Design Handbook (PCI 1992) recommends the inclusion of expansion/contraction joints to account for these types of deflections. Allowing movements at connections of precast members is also valuable for reducing adverse conditions associated with creep and shrinkage deflection.

4-6.6 Storage, construction, transportation, and installation

Proper care of precast concrete elements during storage, construction, transportation, and installation is critical in ensuring that the P/C does not become cracked or permanently deformed. The PCI Design Handbook (PCI 1992) provides guide specifications for the storage of precast panels. There are also suggestions and recommendations regarding lifting and storing. Designing lifting supports near other support areas of the precast elements is recommended and will usually result in the lifting of the element becoming the controlling design case.

If storage of a precast unit/element is designed for three or more supports, then precautions and requirements must be specified so that differential settlement between supports does not occur. Prevention against the warping of panels can be achieved by requiring blocking between panels stacked vertically (or near vertical) and by specifying that panels not be stored in such a manner that the sun hits only one side of the panel during the day.

Provided that the contractor/manufacturer is responsible for the design of the precast elements during stripping, storing, handling, and transportation, the specifications should require them to have a registered Professional Engineer who can approve the calculations and drawings (PCI 1992). Specifying that handling, storage, transportation, and installing should not cause damage, detrimental cracking, or permanent distortion should also be included.

Finally, some methods that will help to maintain the integrity of precast panels are to handle and store them in a vertical or near-vertical position (PCI 1992). For cold-weather climates, storage of P/C should provide protection from water penetration (water and snow) to avoid putting the concrete through freezing and thawing cycles that could damage the structure.

4-6.7 Summary—aids for specifications

The items described above by no means cover everything regarding concerns with the development of specifications for P/C units. As discussed earlier, there is much more detailed information contained in the PCI and ACI documents referenced above. It would also be wise to obtain specifications used on previous projects using precast concrete and speak to those who were involved in the development of the specifications. Pittsburgh District has done specifications for the Braddock Dam float-in structure and Louisville District and Nashville District are currently working on plans that use precast elements and will require specifications regarding these elements.

Precast concrete is a valuable construction method in achieving the float-in and lift-in structures for navigation structures. Careful consideration when writing specifications that are clear and concise will help guarantee that the resulting product will serve its purpose satisfactorily.

5 Precast Loads

Notation

$A_{current}$ = area of floating P/C segment below water
A_i = free surface area of a partially filled compartment
A_{wind} = area of floating P/C segment above water
b = width of the structure
B = water plane inertia
BM = distance from the center of buoyancy to the metacentric point
C = current pressure
C_f = shape coefficient
C_h = height coefficient
d = draft of the structure
D_a = dynamic amplification factor
F = design hook load
F_{steady} = steady loads on mooring lines
F_{surge} = wave surge force due to obstruction of riverflow
g = gravitational acceleration
G = center of gravity
GM = metacentric height
h = depth of fluid or plastic concrete
I = moment of inertia of the water plane
K = reference point on floating segment, often located at the midship on the keel
KB = distance from the keel to the center of buoyancy
KG = distance from the keel to the center of gravity
l = length of the structure
M = metacentric point
p = lateral pressure
P = wind pressure
r_i = distance from free surface of a partially filled compartment to the axis of the water plane of the structure in the direction of rotation

$\mathbf{R}$ = concrete placement rate

$\mathbf{SL}$ = special loads on a lifted object (e.g. tugger line forces, guide forces, and wind forces)

$\mathbf{t}$ = time lapse from initiation of the concrete placement

$\mathbf{t_0}$ = time required for concrete to reach zero slump

$\mathbf{w}$ = unit weight of fresh concrete

$\mathbf{w_{bouy}}$ = buoyant weight of concrete

$\mathbf{W_{lift}}$ = lift weight

$\mathbf{W_r}$ = weight of rigging

$\mathbf{W_{water}}$ = water density

$\mathbf{V}$ = displacement of the structure

$\mathbf{V_{current}}$ = current velocity

$\mathbf{V_{wind}}$ = wind velocity

P/C components are fabricated offsite, and then transported and erected on site to make part of a completed structure. Under various construction and service conditions, P/C components are subjected to different loads that vary in magnitude, direction, and duration. This section identifies various loading conditions that are unique to P/C designs. General structural design load criteria for precast/prestressed concrete structures are described in Chapters 8 through 14.

5-1 Construction Loads

Loads exist during the fabrication of P/C components that must be accounted for in the design. These loads include forming loads exerted on the forms during placement of the concrete and stripping loads due to suction between the concrete and forms and a variety of other factors.

5-1.1 Stripping loads

Stripping loads are determined by self weight of the P/C member, the orientation of the member, impact, suction between the precast product and the form, the number and location of handling devices, and any additional items that must be lifted, such as forms that remain with the member during shipping. In checking the strength of a P/C component against concrete cracking when it is lifted off the form, form suction should be added to the self-weight of the member as an external load. Form suction can be calculated by using a multiplier on the member's self weight, as defined in Table 5.1. A more accurate method is to establish a suction load that is independent of the member's self weight and apply this load over the form's contact area with the P/C component. However, such a method is currently not available.

Table 5.1 Equivalent Static Load Multipliers to Calculate Stripping Loads (PCI Design Handbook 1992)	
Product Type	**Smooth Mold (Form Oil Only)**
Flat, with removable side forms, no false joints or reveals	1.3
Flat with false joints and/or reveals	1.4
Fluted, with proper draft	1.6
Sculptured	1.7

5-1.2 Forming loads

Forming loads are those loads that must be considered during the design of the forms. Design of the forms is usually the responsibility of the contractor, but the designer of the precast structure should at least be aware of what is required for form design.

Forms must be able to provide the support for all vertical and horizontal loads during placement of the concrete without significant deformation. As the concrete sets and increases in strength, the dependency on the forms for structural support is reduced.

Vertical loads that should be considered during form design are the weight of the forms and the weight of the concrete in the forms. These vertical loads will typically not be of great importance if all of the formwork is vertical, but will likely control design for horizontal sections of formwork.

For vertical formwork the most important loading consideration is the lateral pressure exerted by the concrete on the forms when it is initially placed and before it has developed any strength of its own. This lateral pressure can be computed by

$$p = wh \tag{5-1}$$

where

p = lateral pressure, Pa (psf)
w = unit weight of fresh concrete, kg/m^3 (pcf)
h = depth of fluid or plastic concrete, m (ft)

If the full height of the form is filled with concrete during the first placement, then it is equal to full height of the form (ACI 347R). Accounting for the time-dependent reduction in pressure can also be used if needed. This reduction can be accounted for using Equations 5-6 and 5-7 (Section 5-4). Form loads are also a function of concrete temperature, placement rate, concrete workability, and the total duration of the placement. Further information with respect to the effect of these parameters can be found in the "Concrete Forming Design/Construction Guide" published by the APA–The Engineered Wood Association (formerly the

American Plywood Association) (1999). This publication has various design aids that may be used to determine form pressures.

Other horizontal loads that should be included are wind loads for elements that are precast elements, seismic forces, inclined formwork supports, cable formwork supports, concrete dumping, and miscellaneous live loads. Form design shall be in accordance with recommendations given by ACI Committee 347, Formwork for Concrete.

5-2 Handling and Storage Loads

Handling and storage loads are mainly influenced by the orientation of the member, locations of temporary supports and location with respect to other stored members.

5-2.1 Handling loads

The most critical time in handling a precast member is when it is initially lifted from the form. The concrete strength is lower and, in pretensioned members, the prestressing force is higher than at any other time in the life of the member. To minimize concrete stresses due to the eccentricity of prestress, pretensioned flexural members are handled with lifting devices as close as practical to the location where the member will be supported in the structure. With the exception of members with pretensioned cantilevers, lifting devices are located near the ends.

The capacities for lifting devices are generally based on working stress design with a typical factor of safety of 5 against failure (e.g. rupture); the capacities of lifting points for P/C members are generally based upon strength design with typically a capacity/demand ratio of 2 to 3 against limit state exceedance (e.g. yielding of reinforcing steel). There are several reasons for these differences in design for connected elements carrying the same load, some of which are described in Table 5.2.

Concentrically pretensioned or conventionally reinforced members are handled at two or more points, to restrict the concrete tensile stresses below the cracking limit. Normally, a factor of safety of 1.5 is applied to the concrete modulus of rupture. In addition, an impact factor is applied to the dead weight of the member. Optimum lifting locations equalize positive and negative moments in members of constant cross section where the section modulus is the same at the top and bottom. For example, members lifted at two points will have equal positive and negative moments if the lifting points are located at the $1/5^{th}$ point (0.2 times the member length) from the ends. The use of optimum lifting locations is not always necessary, as long as the concrete stresses are within allowable limits. In many cases, available handling equipment determines the lifting locations.

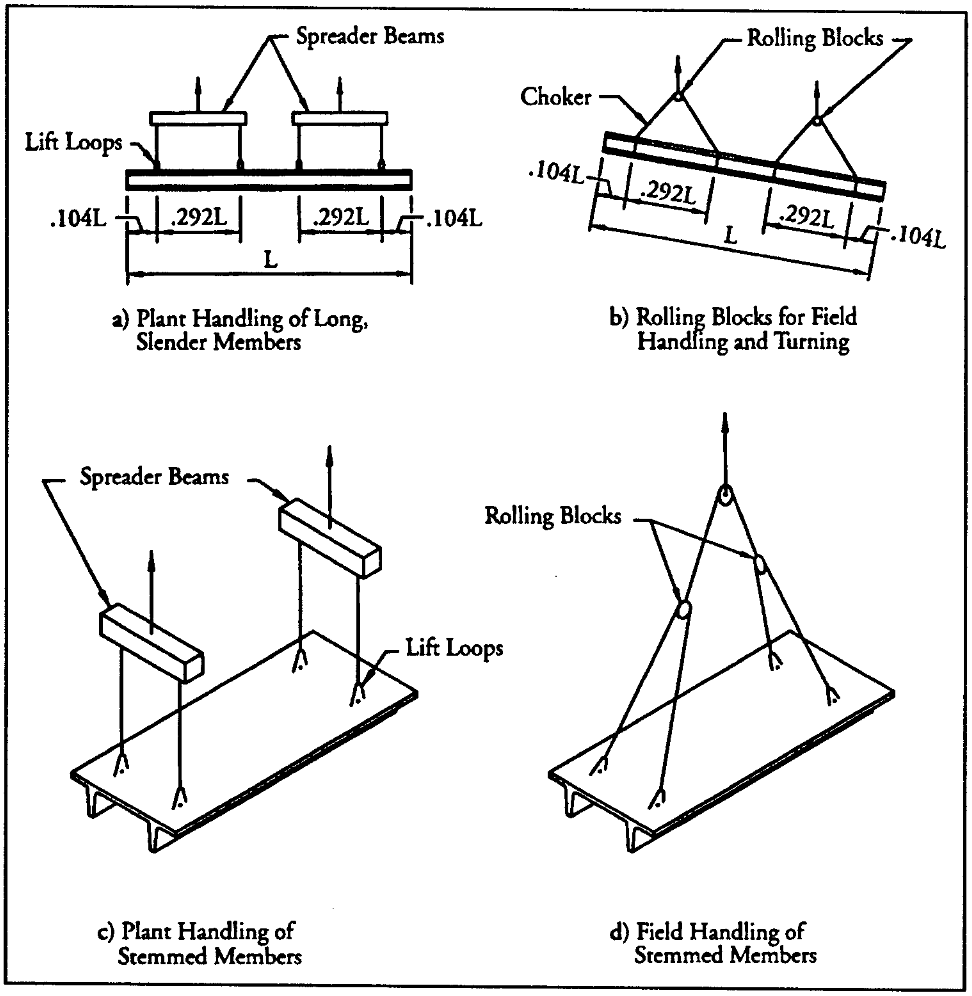


Figure 5.1. Typical lifting devices and arrangements for precast concrete members

Table 5.2
Differences Between Capacities for Lifting Devices and Lifting Points

Design Consideration	Rigging/Working Stress	Lifting Point/Strength Design
Failure mode	Brittle, with no redundancy or reserve strength to rupture	Ductile with reserve strength and potential for load-shedding to rupture
Load characterization	Uncertainty of load characterization, included in safety factor	Individual load factors for different types of loads, including dynamic loading
Resistance characterization	Uncertainty of condition of rigging after reuse, included in safety factor	Individual capacity reduction factors
Potential for wearing during multiple use	Multiple use likely with potential for fatigue and durability concerns	Multiple use unlikely with potential for fatigue and durability checked and addressed in design
Local stress concentration	Included in safety factor	Addressed in design
Likelihood of uneven loading	Included in safety factor	Addressed in design

Long, slender sections can become unstable when handled with lifting devices located near the ends. The most important parameter for lateral stability during handling is the lateral bending stiffness of the member. Inclined slings, such as shown in Figure 5.1b, may introduce significant compression in the P/C member that should be included in calculation of buckling loads. The simplest method to improve lateral stiffness is to move the lifting devices in from the ends. However, doing so normally increases the concrete stresses at lifting and, sometimes, the required concrete release strength.

P/C modules that are not lifted will often be subjected to loads within the prefabrication facility that should be included when performing designs. If a P/C section is moved from a P/C facility on skids, uneven skids can create support conditions that are undesirable due to the variation in the support condition. To alleviate concerns over this, rigid platforms are sometimes constructed so that the varying support does not occur at the base of the P/C module.

Likewise, when a P/C module is launched from a P/C facility, there will be loads that are unique to this condition that must be considered. As a P/C module is launched, it will go through a change in loading due to changes in buoyancy as it enters the water. It is important to stop during the launching process and identify a variety of load cases that occur during the process, to ensure the proper structural capacity during the launching operation.

Further information regarding handling and lifting loads is presented later in this chapter (in Section 5-6) and in Chapter 8. Section 5-6 discusses loads during lifting in more detail, additional load factors that may be required, and other items that are pertinent to the design of lifting points on a precast panel or structure. Discussions in Chapter 8 include information on stages of loading during construction, loading factors, and expected tolerances during lifts.

5-2.2 Storage

Loading of P/C members associated with their storage should be considered during the design process. If storage is to take place on a barge or a slab, the design of the members should consider the locations of the support points. If storage is to occur on soil, the bearing capacity of the soil should be determined. For storage on land or a barge, if tie-downs are used, forces transferred to the structure through these tie-downs must be considered. If the P/C unit is a float-in unit and storage will be on the water, mooring forces that will occur during storage should be included in the design. Loads on mooring lines can be computed using Equation 5-5 (Section 5-4). In addition, for storage on the water, consideration must be given to still-water bending moments acting on the unit while it is moored, as well as any influence from residual stresses on the floating concrete base rafts that may have concrete sections cast on top of them.

5-3 Transportation Loads

A float-in P/C module is subjected to a number of significant loads during its transportation from its prefabrication yard to the project site. The module should therefore be analyzed for its global responses, local responses, and stability. The global responses address stresses and deflections of the entire module in response to external forces as a beam. The local responses include stresses in concrete plates or shells between bulkheads or stiffeners under hydrostatic and hydrodynamic pressures. The floating stability of a floating module is addressed in Section 5-5.

The first type of global loads upon a floating structure is the still-water bending, moment and shear. The buoyancy force distribution usually varies very little along the length of the segment, but the weight distribution is often much more uneven along its length with the presence of concentrated weights, such as those due to pier walls, bulkheads, baffle blocks, and heavy equipment. The disparity between the weight and buoyancy distributions causes the still-water moment and shear in the module. To calculate these forces, the module may be divided into about 20 segments. The net load distribution is obtained by taking the difference between the cumulative weight and buoyancy forces from these segments. The shear and moment distributions are then the first and second integrals of this net load.

Besides the still-water condition, the sagging and hogging conditions caused by waves can also induce significant forces in float-in modules. When the weight distribution of a floating structure is closely matched by the buoyancy force distribution, dynamic wave and wind loads may constitute 80 percent or more of the design loads. Figure 5.2 is a sketch of a floating module in the still-water condition, a sagging wave condition, and a hogging wave condition. The lower part of the figure shows the weight distribution and buoyancy distribution correspondence to the three cases. The sagging condition usually exacerbates the load effects of the still-water condition, while the hogging condition reverses the load effects of the still-water condition.

To assess the sagging and hogging wave loads, design criteria should specify height and length of the design wave induced by river current, wind, and passing vessels. If the float-in module will remain afloat during its construction and outfitting for a substantial period, the design should include the design wave condition pertinent to the construction site. Structural calculations must be performed to determine the sagging and hogging components. These additional forces are then added to the still-water moments and shear. The wave action must be checked in both the longitudinal and transverse directions of the float-in module as well as quartering waves. Any significant torsional moment induced by the design waves should also be included in structural calculations. These wave-induced forces are applied to develop the maximum bending moments and shears in the precast segments, since they depend on the orientation of the vessel as well as the wave length and height. The design wave can be computed using the fetch, the wind, and the duration using manuals published by the Navy, the American Bureau of Shipbuilders, or the Society of Naval Architects and Marine

Engineers (Comstock 1967). It should also be noted that the work described in the previous paragraphs is typically performed by a naval architect.

All the environmental forces should be determined and included in structural calculations. The design loads may be based upon actual measurements at the site. In the absence of field-measured data, the following formulas are used to calculate wind, current, wave drift, and wave force:

$$P = 0.6105V^{2}_{wind} C_h C_f \tag{5-2}$$

where

P = wind pressure in Pa (psf)
V_{wind} = wind velocity in m/s (ft/sec)
C_h = height coefficient as defined in Table 5.3
C_s = shape coefficient as defined in Table 5.4

Table 5.3
Values of the Height Coefficient C_h

Height Coefficient Value	Height from the Water Surface to Center of Design Surface Area			
	0-15 m (0-50 ft)	15-50 m (50-100 ft)	30-46 m (100-150 ft)	46-61 m (150-200 ft)
Coefficient C_h	1.00	1.10	1.20	1.30

Table 5.4
Values of the Shape Coefficient C_s

Shape of the Components	**Shape Coefficient Cs**
Hull	1.0
Deckhouse	1.0
Isolated structural shape (cranes, angles, channels, beams, etc.)	1.5
Cylindrical shape (all sizes)	0.5
Rig derrick	1.25

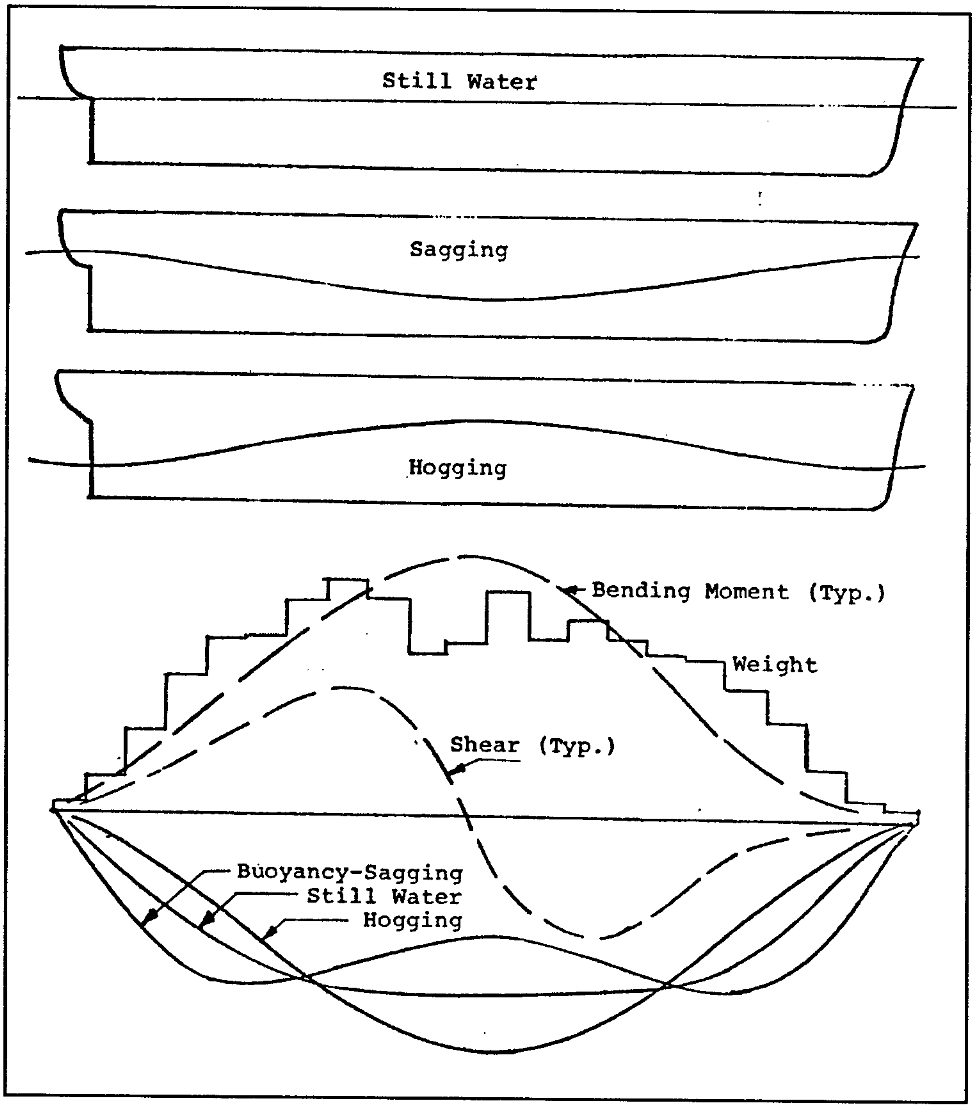


Figure 5.2. Still-water, sagging, and hogging conditions; shear force and moment distributions under the still-water condition; buoyancy of still-water, sagging, and hogging conditions

Current pressure (C), in pascals, or pounds (force) per square foot, is defined by

$$C_s \frac{W_{water} V_{current}^2}{2} \tag{5-3}$$

where

C_s = shape coefficient
W_{water} = water density in kg/m^3 (lb/ft^3)
$V_{current}$ = current velocity in m/s (ft/sec)
g = gravitational acceleration in m/s^2 (ft/sec^2)

The resultant environment load is

$$F = PA_{wind} + CA_{current} \tag{5-4}$$

where

A_{wind} = area of segment above water
$A_{current}$ = area of segment below water

The towboat will be required to overcome bow waves, effect of static drift forces, wind, and current. The influence of the pull force and wind loading on the draft is considered negligible. While the pull force of the towboat on the draft is negligible, the load imparted to the P/C unit must be considered in the design. This force is a localized force and, therefore, the design will be for a localized area. The information needed to perform the design should be obtained from a naval architect. In addition, dynamic amplification of the structure due to waves should be considered during the design. Further discussion on the design of float-in units during transportation is presented in Section 8-1a.

Contingency mooring should be provided along the route. This contingency mooring should generally be based on the expected transit time of the float-in unit and the ability to forecast weather and severe flood events. For example, for smaller rivers that can rise quickly and for which forecasting is limited, moorings along the route should be provided if the transportation of the precast module takes more than 24 hr. However, for larger rivers for which predictions can be made based on expected rainfall, contingency moorings may not be needed. Contingency mooring should also be provided at the outfitting site if the site will be exposed to flooding. Again, for small rivers that rise quickly, the contingency moorings should be designed for at least 100-year flood. Single-point moorings are typically used for the contingency mooring line, along with an anchor to prevent yaw. The magnitude and heading of wind, currents, waves, and dynamic excursions make it difficult and risky to rely on two mooring lines acting efficiently at the same time.

Lift-in modules require certain considerations during transport as well. If a lift-in module is transported to the site on a barge, the support conditions during the transporting phase must be given due consideration. Often, these will be very similar to conditions used for storage. Forces due to wind and rocking action that may occur must be included. An important factor of the design for transportation of lift-in units on barges is ensuring that the design allows for proper clearances from the prefabrication facility to the construction site.

When considering the transportation of a lift-in module to a project site, consideration should be given to how the module will be removed from the barge. Options include lifting it with a conventional crane, lifting it with a catamaran, sliding it off the barge, and submerging the barge. These considerations are important because it will have an impact on the selection of the barge used to transport the module as well as the loadings the module will experience.

For lift-in modules that are transported to the site partially or fully submerged, loadings that must be considered are more extensive than those transported on barges. Buoyancy forces will be a factor, as well as forces due to the current of the river. Chapter 8 provides a more detailed discussion of the forces that should be considered during the design process.

5-4 Erection Loads

For in-the-wet construction, typical erection loads on a P/C module include mooring line forces, ballasting loads and/or lifting forces, infill concrete pressure, pressure from thermal expansion of infill concrete, concentrated loads from support points, environmental loads such as current and wave drift, and wave surge forces.

Mooring line force – During erection of a precast module, a mooring system is normally used to position and hold the segments while they are being lowered by a crane or ballasted down to riverbed. The mooring force should be a sum of steady (static) loads and transient (dynamic) loads. The steady loads are the wind, current, and wave drift forces discussed in Section 5-3. Furthermore, for large waves there is a significant surface wave surge when a large segment is set on the riverbed and extends over the water surface. The wave surge is due to obstruction of wave flow by the segment. The wave surge force and the resulting overturning moment should be included in structural calculations by using proper theories such as the nonbreaking wave formula in the Shore Protection Manual (U.S. Army Corps of Engineers 1984). The resultant steady loads on mooring lines are as follows:

$$F_{steady} = PA_{wind} + CA_{current} + F_{surge} \tag{5-5}$$

where F_{surge} is the wave surge force due to obstruction of riverflow.

Transient loads include vertical wave load causing a vertical excursion of the modules that will stretch the mooring lines. In general, the transient load is considered only for unusual conditions, such as major flooding.

The resultant mooring line loads are calculated without any load factors. The safety factors are built into the mooring line and anchor pile capacities. A safety factor of 5 to 6 is normally used on the breaking strength of mooring lines, although the Corps' Safety and Health Requirements Manual (EM 385-1-1, HQUSACE 1996) requires a minimum factor of safety of 5 for shackles. This is assumed to compensate for wear, impact, combined stresses at bends over sheaves, and material uncertainties.

Infill concrete pressure – In-the-wet construction may utilize P/C modules as in situ forms for infill tremie concrete. As the infill concrete is being placed into the forms, the hydrostatic concrete pressure on the forms increases. The form pressures in combination with the thermal expansion of the infill concrete may dictate the design of the precast modules. Tremie concrete pressure on the precast modules should be adequately designed in accordance with EM 1110-2-2104 and Technical Report INP-SL-1. In principle, the magnitude of the form pressures

primarily depends on the rate of the concrete placement and the rate of the tremie concrete slump loss. If placement of concrete is slow enough to allow the concrete at the bottom to stiffen, the form pressure will correspondingly decrease. A simple design method is to assume liquid pressure of the concrete without considering the time-dependent reduction of the form pressure, as described in Equation 5-1. However, in some cases, neglecting the reduction of the form pressures can result in uneconomical design of the precast module. As an alternative, a bilinear pressure diagram, as shown in Figure 5.3, may be used to obtain the design form pressure. In other words, the time period required for the concrete to reach zero slump is recorded as t_0. The form pressure is calculated as follows:

$$p = w_{bouy} * R * t \qquad \text{when } t < t_0 \qquad (5\text{-}6)$$

$$p = p_{max} = w_{bouy} * R * t_0 \qquad \text{when } t > t_0 \qquad (5\text{-}7)$$

where

w_{bouy} = buoyant weight of concrete in kg/m^3 (lb/ft^3)
R = placement rate in m/hr (ft/hr)
t = time lapse from initiation of the placement, hr
p_{max} = maximum form pressure in Pa (psf)
t_0 = time for concrete to reach zero slump, hr

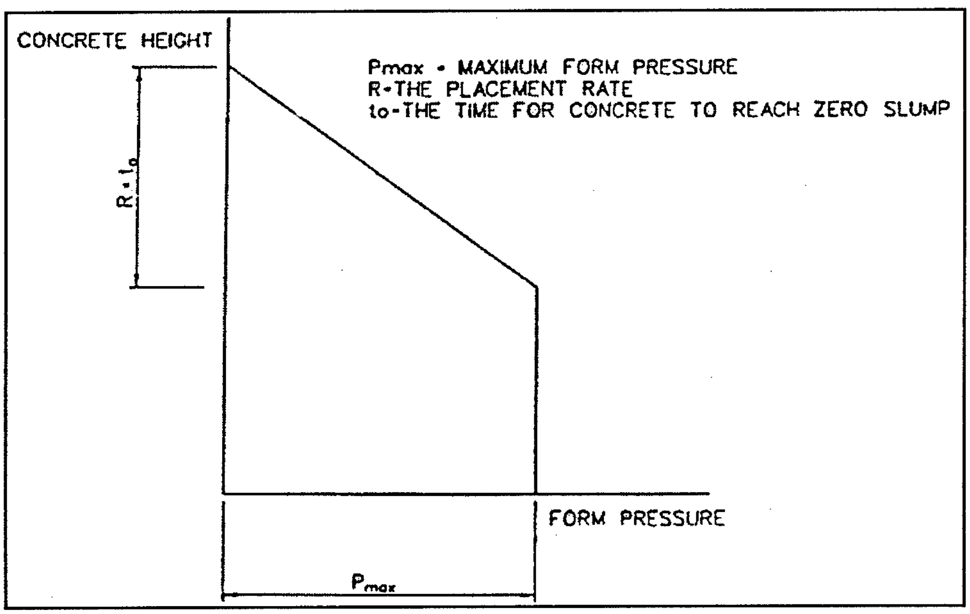


Figure 5.3. Design diagram of underwater concrete form pressure

More detailed information regarding infill concrete pressure can be obtained from ERDC Technical Report INP-SL-1 (Yao, Berner, and Gerwick 1999).

Thermal loads - The thermal expansion of infill concrete due to heat of hydration is one of the critical problems for in-the-wet construction of navigation structures. The thermal expansion of mass concrete and steep temperature gradient in the P/C form can lead to unacceptable cracks in the precast concrete. Thermal loads may be estimated with an approximate method, such as the

Schmidt method as described in ACI Committee 207.1 report. For major projects, it may be necessary to perform a full suite of material tests for the infill concrete, including the adiabatic temperature rise test, and a nonlinear incremental stress analysis (NISA), in accordance with Engineer Technical Letter (ETL) 1110-2-365 (HQDA 1994), to determine the thermal load and its effects. Further discussion regarding the performance of a NISA is presented in Chapter 7.

Superimposed additional loads – Once a P/C module is set down on its supports or foundation, additional loads, such as lifting frame, equipment, differential settlement, and additional P/C components, are likely to be imposed upon the module. Unless the infill concrete reaches the design strength and acts compositely with the P/C module, the module should be designed to carry all the imposed loads.

5-5 Ballasting

Ballasting of a float-in P/C module may be performed at different construction stages for various purposes. Ballasting during launch or during outfitting is usually performed to trim or to lower the center of gravity for required floating stability. Ballasting is also performed at the project site for the purpose of setting down the segment onto a prepared foundation.

Ballasting of a floating module is performed by filling its interior compartments with either solid or liquid ballast. Concrete may be used for permanent solid ballast. During all phases of transport and outfitting, water ballast is normally used for minor trimming of the floating modules. Water is the primary ballast material during the setdown operation. Gravel or sand may also be used for ballast. Iron ore has been used where high-density ballast is required.

The staging of the ballast sequence and the amount of ballast at each stage shall be specified in the design. The layout of compartments and sequence of ballasting shall be such as to reduce the overall stresses in the structure by filling the compartments in a pattern that results in a more uniform distribution of the dead loads. All compartment bulkheads and keel plates as well as the entire floating module shall be designed to withstand the ballast loads at different stages of ballasting. Should loadings during the ballasting sequence create excessive bending and flexural stresses, the use of an external support system (e.g. a steel frame) may be considered to better distribute the loads and neutralize these stresses.

It should be noted that if ballasting is occurring during the winter season, the ballast water might freeze. Freezing of the ballast water should be carefully considered prior to beginning the ballasting sequence since weight shift due to water moving in the compartments may not occur.

The stability of the module during ballasting can be determined using the procedures outlined in Section 5-9 on floating stability.

5-6 Lifting Loads

The lift-in P/C modules cannot float by themselves. Usually, towed barges or floating cranes are used to transport and place the modules. The lifting operations of a crane do not represent one well-defined load case, but rather a sequence of different load cases. Uncertainties with respect to internal force distribution and possible accident loads require a high safety margin. The designer should in principle consider the entire lifting sequence step by step and identify the most critical dynamic load case for each structural member. The dynamic effects of lift operations can be influenced by a number of factors, such as environmental conditions, motions of crane barges, stiffness of lifting gear/equipment, lift weight, and whether lift is in air or water. The dynamic effects are mainly due to acceleration of lifted objects. The dynamic effects may be significantly increased when lifting a heavy load off a barge in a wave environment, since the barge may drop suddenly before the crane has lifted the load.

In lieu of refined analyses, the dynamic effects may be included in calculations by means of dynamic amplification factors, as defined in Table 5.5. An alternative method for calculating the dynamic amplification factor is given in Section 8-1.c(2).

Table 5.5
Dynamic Amplification Factor of Calculating Lifting Loads

Lift Weight	<90.7 Mg (<100 tons)	90.7 – 907 Mg (100 - 1,000 tons)	907 – 1814 Mg (1,000 - 2,000 tons)	>1814 Mg (>2,000 tons)
Dynamic amplification factor	1.15	1.10	1.05	1.05

The basic design load case is the dynamic hook load. The design lift hook load shall be calculated as follows:

$$F = D_a(W_{lift} + W_r) + SL \tag{5-8}$$

where

F = design hook load
D_a = dynamic amplification factor
W_{lift} = lift weight
W_r = weight of rigging
SL = special loads, such as tugger line forces, guide forces, wind forces on the lifted object

To design the precast modules for the lift loads, a load factor of 1.7 shall be applied as a multiplier to the design load to account for uncertainties of load inaccuracy, local dynamic effects, and consequences of failure.

One of the critical components is the connection of the lifting gear to the lift-in precast segments. The gravity loads from the segment have to be picked up by the lifting gear. Similarly, the lifting forces need to be distributed to the precast segment lifting points, and their attachments to the precast modules are to be

designed for the maximum lifting load plus any lateral load component. Typically, the maximum negative moments occur at the attachment points. To prevent excessive cracking in the segment, additional strengthening and reinforcement confinement around the embedment of lift device (padeyes) are often provided. For odd-shaped precast segments, torsion must also be considered. Padeye plates should be oriented in such a direction that the possibility for out-of-plane loading of the padeye plate and on the shackle is minimized.

Precast concrete modules can be subjected to very high bending and tensile stresses near the picking points during lifting operations. Therefore, the flat plate modules are usually fitted with a structural steel frame to facilitate the lift-in operation. The steel frame is usually secured to the top of the module before launching. The frame can serve multiple purposes. First, it distributes the lift force from the crane hoist to the concrete segment through many picking points, thereby reducing bending moments in the segment. Second, the frame serves as a spotting tower for accurate positioning of the segment under water. And third, it may serve as a guide frame for lowering tremie pipes into the segment at specific locations so that underwater concrete can be placed. Thus, the steel frame can also be referred to as the tremie support frame.

The design for lifting P/C panels is a cooperative effort between the designer and the contractor. The designer should include picking points in the P/C panel in his design. However, he should be ready to review alternative picking points should the contractor determine that there might be a more optimum configuration for lifting the panel. If it is known during the design that a frame will be used in lifting a P/C panel, the designer should include the design for this frame in his calculations. The contractor will always be responsible for the design of the rigging used to lift panels.

5-7 Final Loads

The loads that are expected to be imparted to navigation dams, locks, and their appurtenant structures include hydrostatic pressure due to differential elevations, uplift, lateral earth pressure, tow impact, hawser loads, ice and debris, wave and wind loads, bulkhead and gate loads, seismic loads, thermal loads, superstructure and equipment loads, sheet-pile or cofferdam tie-in loads, and loads acting within the monolith joints. Design loads on navigation dam structures should be specified in accordance with EM 1110-2-2607. Design loads on navigation lock structures should be specified in accordance with EM 1110-2-2602. In addition, due consideration needs to be given to the cyclic loads to which the structure may be subjected. These types of loadings should be considered carefully with respect to the fatigue capacity of the structure.

5-8 Progressive Failures

In addition to checking stability of a floating structure, structural engineers should also check its strength against additional forces and pressure when the

structure experiences significant trim/heel or accidental local damage. For permanently floating structures and the transport of float-in structures, the progressive collapse limit state (PLS) should be checked against catastrophic failure. PLS corresponds to the condition that failure of one member due to accidental overloading leads to progressive failure of adjoining members. The best approach for determining a PLS would be to incorporate a risk analysis into the progressive failure of the structure. The PLS design is usually achieved by a combination of the structural redundancy and ductility. At the element level, individual members should be designed for adequate ductility. At the level of the structural system, compartments provide contingency against accidental flooding during float-out and transport. All walls behind the external walls should be watertight. A float-in structure is commonly designed for flooding of one perimeter compartment. A permanently floating structure is commonly designed for simultaneous flooding of two compartments without significant impact on its stability and strength; that is, multiple load paths are provided in all critical regions.

5-9 Floating Stability

Floating structures and float-in modules must meet the design requirements for floating stability. In essence, the structures should be able to remain floating upright for all afloat conditions, including launching and ballasting down, and under all the possible environmental conditions pertaining to the site and the period. They should also have adequate reserves of stability when certain accidental damage occurs.

A floating structure may lose its stability due to several destabilizing effects, such as flooding of its compartments or lifting loads from a mounted crane. A stability check according to naval architecture principles should be made against all the potential destabilizing effects.

Three important parameters control the stability of a floating structure: the center of gravity (G), the center of buoyancy (B), and the water plane inertia (I), as shown in Figure 5.4. A reference point is often established at midship on the keel (K). When a floating structure heels or trims, the buoyancy force acts vertically upward through B to intersect the axis of the structure at the "metacentric point" (M), as shown in Figure 5.5. The buoyancy force also imposes a righting moment on the structure. The righting moment is the product of the displacement and the righting arm, ($\overline{GM}$)(sin θ), where $\overline{GM}$ and θ are shown in Figure 5.4. For small angles of list, sin θ may be replaced by θ. Stability of the structure requires that the righting moment restore the structure to the upright floating position once external forces causing the heel or trim are removed. In naval architecture, this stability requirement implies that the metacentric height should always be positive. In practice, the metacentric height ($\overline{GM}$) is usually kept above 1 m (3.3 ft) for all directions of inclination. This stability requirement can be translated into the following mathematical equation:

$$\overline{GM} = \overline{KB} - \overline{KG} + \overline{BM} \geq 1.0\, meter \tag{5-9}$$

where *KB* and *KG* are the distance from the keel to the center of buoyancy and the center of gravity, respectively. The distance from the center of buoyancy to the metacentric point (*BM*) can be calculated as follows:

$$\overline{BM} = \frac{I}{V} = \frac{Moment\ of\ inertia\ of\ the\ waterplane}{Displacement\ of\ the\ structure} \tag{5-10}$$

For any rectangular structure, $I = \frac{b^3 l}{12}$ and $V = bld$. Variables *b*, *l*, and *d* are the beam, length, and draft of the structure, respectively.

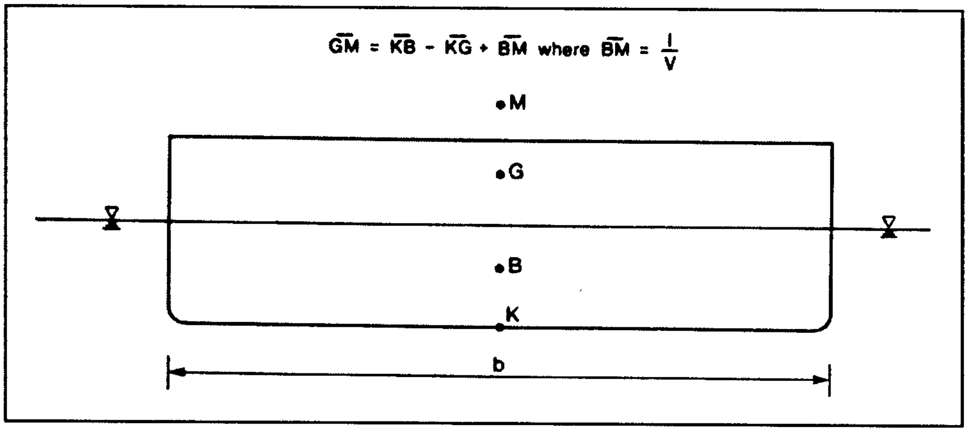


Figure 5.4. Center of buoyancy, center of gravity, and metacenter

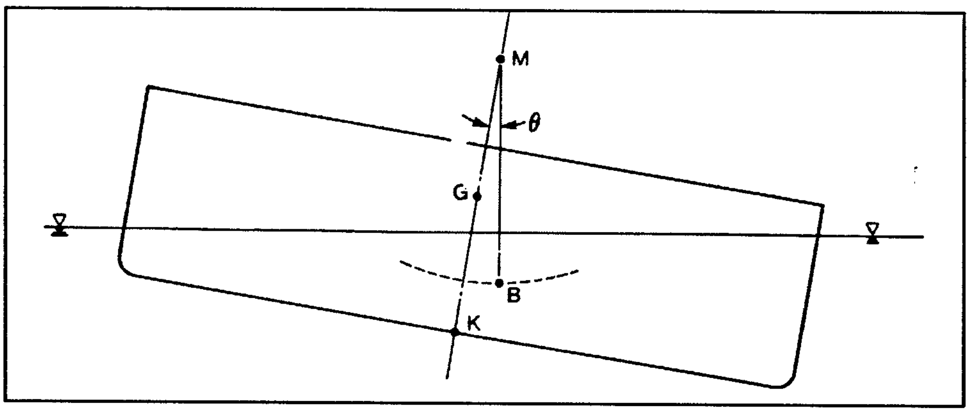


Figure 5.5. Effect of metacentric height

Floating stability shall be checked for all the possible cases of flooding. There are a number of reasons for flooding a floating structure, e.g., boat impact, incorrect valve operation, or ballasting down a float-in module. The principal method of controlling stability is to subdivide a floating structure into a sufficient number of small compartments so that accidental flooding is limited to a small part of the structure.

If one or more compartments are partially filled with water or other liquid, the internal water planes will cause a shift of the center of gravity farther away from the center of buoyancy upon heel or trim of the structure. The net effect,

often referred to as "free surface effect," is a reduction in stability and the metacentric height. The free surface effect can be approximately accounted for by subtracting its contribution from BM as follows:

$$\overline{BM} = \frac{I}{V} - \sum_i A_i r_i^2 \tag{5-11}$$

where A_i is the free surface area in a partially filled compartment, and r_i is the distance from the free surface to the axis of the water plane of the entire structure in the direction of rotation.

Some float-in structures have partially or fully open tops during transport. Adequate consideration must be given to the potential for overtopping, such as by waves or due to unintended list, and even to rain water, because even a small amount of floodwater can lead to significant free-surface effects.

To provide protection against accidental flooding, the damage control measures require that all manholes, hatches, and bulkheads in the compartments be sealed watertight and designed to withstand the maximum pressure head of the accidental flooding. The design also requires that all pipes and ducts be closed off during nonusage periods so that flooding does not spread through those systems.

In ballasting a float-in segment down to riverbed, it is critical to check the stability when the segment commences immersion into water. Since the waterline plane diminishes at that instance, the moment of inertia diminishes rapidly. Special caution must be taken to ensure adequate metacentric height (at least 1 m) for the stability of the float-in segment in all directions of inclination. During submergence, if the segment is ballasted down uniformly so that the top deck becomes awash at approximately the same time, the segment will lose a significant portion of the water plane and may behave unpredictably. Therefore, it is good practice to purposely tilt the segment in one direction (usually in the longitudinal direction) so that the water plane is reduced gradually while the mooring lines and guide device hold the segment in place.

To check the stability of a float-in segment during submergence, further calculations should be made to take into account the dynamic stability of the segments during final immersion, the effects of sloshing of the free surface water on the overall stability, the effects of mooring lines, and the effects of pressure reduction due to increased velocity under segments during final stages of setdown.

Lifting loads from a floating structure can substantially change its stability condition. In stability calculations, the weight of the loads should be assumed to act at the height of the upper support point (e.g., the sheave blocks at the tip of the crane boom). In such cases, the center of gravity can be very high. The stability becomes excessively sensitive to the instantaneous transverse moment of inertia.

The above formulas are useful tools for quick assessment of the hydrostatic stability of a floating structure. They are valid for small heel/trim rotational

angles. If a floating structure experiences substantial trim/heel rotation, stability calculations should be based upon the righting moment stability criterion, as specified in "Rules for Building and Classing Steel Vessels for Service on Rivers and Intercoastal Waterways" and "Rules for Building and Classing Mobile Offshore Drilling Units" (American Bureau of Shipping 1985, 1980). This approach is also described in EC 1110-2-6052 (HQDA 2001) and in "Principles of Naval Architecture" (Comstock 1967). However, it should be noted that these calculations should typically be performed by a naval architect.

5-10 Load Cases

Because the nature of float-in and lift-in construction is significantly different from the conventional methods that the Corps of Engineers has used in the past, additional load cases will be needed during the design process to ensure that the P/C units are not overstressed or damaged prior to erection. Some load cases are given below to assist in summarizing the loads and their combinations during the design process.

Precast yard loads:

- Dead weight of concrete
  - During casting, prior to form removal
  - On yard skids (with dynamic factor)
- Dead weight of forms
- Lateral pressure of concrete
- Form suction
  - When lifting from casting bed
  - Between concrete and forms when vertical forms are removed
- Post-tensioning jacking force for anchorage design

Handling:

- Dead weight of concrete
- Impact factor
- Lateral loads due to lifting devices (slings)
- Hydrostatic pressure
- Ballasting loads
- Dynamic inertial/drag (current)

Storage:

- Dead weight of concrete
- Wind loads
- Seismic (for storage on land)
- Support conditions (for storage on barge or land)
  - Possible uneven support
- Mooring line loads (for storage on water)

Transportation:

- Dead weight of concrete
- Cumulative weight versus buoyancy forces (see Section 5-3)
- Hogging
- Sagging

- Wave action (including dynamic amplification)
- Wind
- Current action
- Forces from tow lines
- Ballasting
- Impact/collision

Erection:

- Dead weight of P/C concrete
- Mooring line forces
- Ballasting loads (float-in)
- Lifting forces (lift-in)
- Pressure of infill concrete
- Thermal loads due to infill concrete
- Other thermal loads
- Initial support conditions
    - Point loads at jacking supports
    - Uneven foundation
- Current drift forces
- Wave drift forces
- Wave surge forces
- Equipment loads
- Weight of the lifting frame

Operation:

- Dead weight of concrete
- Earth pressures
- Hydrostatic pressure
    - Normal pool/tailwater
    - Maximum probable flood
- Uplift
- Earthquake
- Barge impact
- Wind
- Ice
- Machinery
- Prestressed or post-tensioned loads

6 Connections

Notation

h = thickness of the member

In terms of their structural functions, joints in hydraulic structures may be divided into moment connections, hinged connections, expansion/contraction joints, and isolation joints. Off-site prefabrication and underwater installation of P/C modules implies that loads are generally transferred through a discrete number of connections, which may be more critical than those in the conventional cast-in-place construction. Connections to tie superstructures into pile foundations and to join precast segments into a monolith are of paramount importance. Both temporary and permanent connections must be designed with careful attention to details and construction procedure to ensure the critical load paths and durability of performance. Permanent connections have been designed for both temporary and permanent load cases. However, it may be a more economical design approach to use temporary connections that are not required in the permanent condition. A more detailed discussion of connections of large units can be found in ERDC/GSL Technical Report "Modular Connections and Seals for Precast Concrete Segments" (McDonald, in preparation).

The PCI Connection Manual MNL-123-88 (PCI 1988) notes that the majority of connection failures in precast construction are the result of improper design for volume change associated with temperature and shrinkage. A careful evaluation of temperature- and shrinkage-related volume change in connections is required for both temporary and permanent load conditions. An example of temporary loadings would be temperature changes associated with taking precast elements that have been stored in a sunny area and placing them in cold water or hydration temperatures associated with placement of tremie concrete within a precast shell. Permanent loads could include seasonal temperature variations or differential settlement of foundation elements. Where filler material other than concrete or grout is used in connections, the coefficient of thermal expansion for the filler material may vary from the precast element, resulting in additional thermal loads due to differential volumetric changes.

In addition, MNL-123-88 notes the following items to consider when designing connections:

a. Standardize connection types.

b. Avoid reinforcement and hardware congestion.

c. Avoid penetration of forms where possible.

d. Reduce post-stripping work.

e. Be aware of material sizes and limitations.

f. Consider clearances and tolerances.

g. Avoid nonstandard production and erection tolerances.

h. Standardize hardware items and use as few sizes as possible.

i. Use repetitious details.

j. Use symmetrical connection materials to minimize errors.

Congested reinforcing can be a problem in connections of precast thin-wall structures, as shown in Figure 6.1, from the Braddock Dam Project. These connections often contain a large amount of reinforcing steel within a relatively narrow space. The designer should draw details to scale, including bar bend radius, diameter of reinforcing, anchor head size, cover, and other embedded items to ensure that conflicts are not present. Thin-wall concrete structures have similar (congested) profiles to post-tensioned or precast bridges. Many bridge designers are requiring 3-D drawings of concrete bridges to show ducts, reinforcing, embedments, bar supports, and ties to avoid conflicts during construction. An alternate is the use of a mockup for complicated connections. The 3-D drawing or mockup can be submitted by the contractor prior to full-scale production.

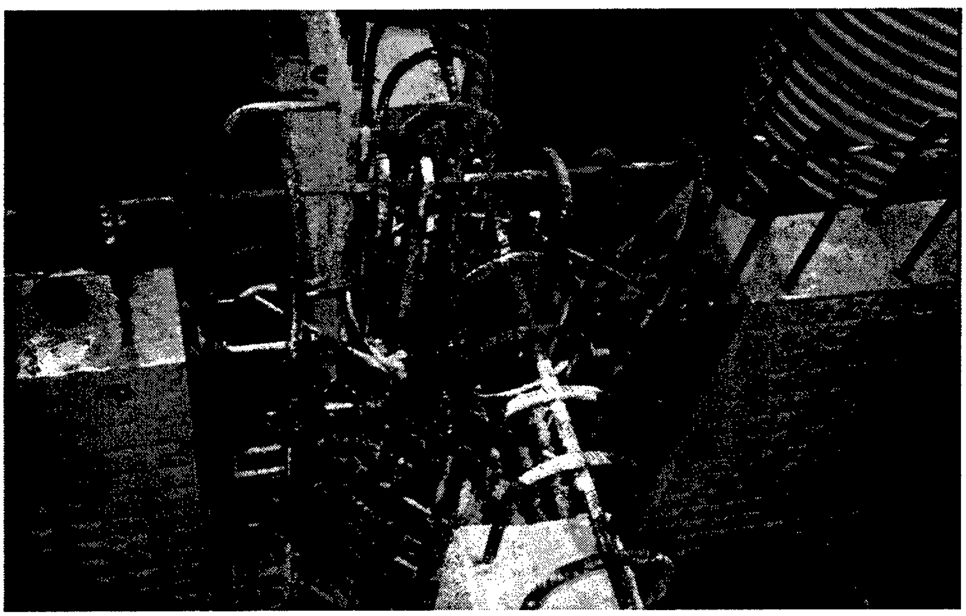


Figure 6.1. Congested reinforcing arrangement on Braddock Dam float-in segment

The designer should become familiar with reinforcing fabrication guidelines such as the "Manual of Standard Practice" prepared by the CRSI (Concrete Reinforcing Steel Institute 1990) or the ACI Detailing Manual, which discusses detailing of reinforcement. CRSI provides some recommended placement

sequences for mat and slab construction. The designer should consider reinforcement placement sequences when detailing complicated connections.

Additional factors related to congestion of reinforcing include

- *Hoops:* Connecting two panels that terminate horizontal reinforcing in closed hoops will require the hoops to miss each other. If vertical reinforcement is detailed inside the hoop, then any vertical reinforcing bar will have to be installed by threading the bar through the hoops.
- *Headed terminations:* The use of headed terminations to reduce development length can be useful in small spaces. However, the heads can interfere with adjacent reinforcement, make concrete placement more difficult, and create a preferential cracking plane along the head (for tight spacing).
- *Field tolerances:* The designer should consider allowable field tolerances when evaluating interferences.
- *Long reinforcing runs:* Connections that require long runs of reinforcing are more problematic since the bars may have to thread through other reinforcement over the long bar length.
- *Bar bends:* Allowable bending radius for large bars often results in interferences. For example, a No. 11 bar requires a bend radius of 357 mm (14 in.).

6-1 Monolithic Action

The design of all hydraulic concrete structures that incorporate P/C components in their construction shall be based on the philosophy that the completed structure will act as a monolithic structural system. To ensure overall ductile behavior of the monolith, connectors such as deformed bars, headed studs, or anchor bolts shall be designed with sufficient embedment to ensure that failure of the reinforcement by yielding occurs prior to failure of the concrete. For large structures with few connections, load transfer frequently requires extensive patterns of anchors or reinforcement. The design should consider group effects as well as individual capacities. A discussion of the design of large groupings of anchors is contained in Kuchma, Tamas, and Collins (1999).

Specific joint details shall be developed to accomplish monolithic action. Connection details for moment connections shall ensure full transfer of moment, shear, and axial forces across the joints between adjacent components. Connection details for hinged connections shall ensure full transfer of shear and axial forces. Connection details for expansion/contraction joints shall ensure full transfer of shear. Connection details for isolation joints shall ensure no transfer of force and moments across the joints, and allow free movements of adjacent components.

This design approach requires the structural engineer to coordinate the joint design with cost estimators, and construction field personnel to develop cost-optimized P/C systems. Although details contained in PCI (1988) or developed

by local precasters are typically intended for building construction, they can be adapted for use in construction of hydraulic structures. Energy-absorbing designs for P/C structural systems (nonmonolithic joint details) may have economical applications in seismically active regions. Designs based on this philosophy require approval from Headquarters, U.S. Army Corps of Engineers (USACE).

6-2 Clearance

Clearance is defined as the space required between elements to allow proper assembly of the elements. Precast connections generally require greater clearance than cast-in-place construction. Clearance requirements should include tolerances as well as providing sufficient room for tools (welding, torque wrench), access to connection, ability to clean the connection, and ease of construction. Typical clearance recommendations from PCI (1985) are contained in Table 6.1. Both fabrication tolerances and erection tolerances should be evaluated when determining the required clearance. Further discussion of recommended tolerances is contained in Appendix C. A more complete discussion of this subject is presented in a journal article co-authored by ACI and PCI writers (PCI 1985).

Table 6.1
Recommended Clearances

Item	Recommended Minimum Clearance, mm (in.)	Preferred Minimum Clearance, mm (in.)
Precast to precast	12.7 (0.5)	25.4 (1)
Precast to cast-in-place	25.4 (1)	50.8 (2)
Precast to steel	25.4 (1)	50.8 (2)

Erection tolerances contained in Appendix C are general references for construction of P/C components in the dry condition. They are not suitable for establishing erection tolerances for underwater positioning of large P/C modules. These recommendations were developed to minimize the impacts of misalignment on the structural performance and esthetic appearance of the structures. In particular, the use of larger erection tolerances for underwater construction may require consideration of the impacts of misalignment and eccentrically applied loads on the overall design of the connection.

Several connection details have been developed to allow the connection of precast elements with large tolerances. A brief discussion is given in the paragraphs below.

Slotted steel plates can be used to connect precast elements (such as precast armor) to support frames. The slots allow some movement of the panels to accommodate installation tolerances. Where slotted connections are used to allow slip movement in one direction, overtightened bolts will result in a slip-critical connection that may require significant loading prior to movement of the connection. This is more likely to occur during erection where bolts may be tightened during fit-up; however, this loading could occur in a permanent condition.

Leveling nuts are often used on anchor bolt connections to column base plates. This technique could be adapted to other underwater connections. The nuts are installed several inches above the concrete surface and are adjusted to level the bearing plate when anchor bolts are out of plumb. The nuts on top of the plate clamp it to the leveling nuts. A dry-pack grout pad or tremie concrete pad is installed under the plate. Loads due to eccentric loading of the bolts by the plate should be considered in bolt design.

Mortise connections are often used to match plate elements in precast building construction (Figure 6.2). The plates lap one another and are connected in the field using pins. Threaded inserts, mechanical couplers, and other mechanisms for connecting reinforcing can also be used with this method. Once the element is erected into place and adjusted for field tolerances, concrete is placed in the voids of the connection to create a monolithic structure. The connection of the Braddock Dam float-in structure to the drilled shaft foundation consists of modified mortise connection where corrugated steel pipes are sealed at the top to form an upside-down mortise (cap) that fit over the tops of the shafts. The structure is leveled, and the steel pipe is grouted to complete the connection.

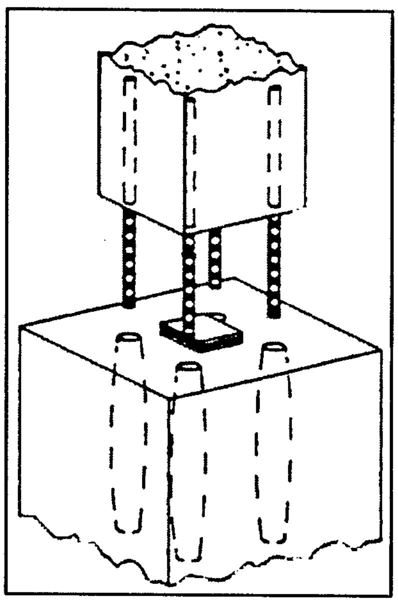


Figure 6.2. Mortise connection

Collar connections consist of cast-in-place "collars" that are poured around the connection of two precast elements, as shown in Figure 6.3. In general, these connections are considered hinge connections and most likely will not provide a watertight joint.

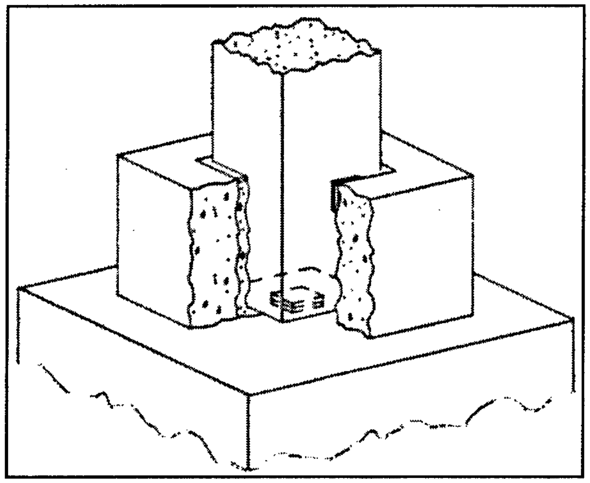


Figure 6.3. Collar connection with cast-in-place collar

Through-bolt connections are used to connect plate and wall elements in a butt joint (Figure 6.4). Bolts are installed through pockets left near the ends of the plates. Sufficient room to tighten the bolts must be provided in the pockets. The through-bolts can be used to snug up the ends of a butt joint. Sufficient concrete must be provided between the pocket and the end of the member to develop the capacity of the bolts. The pockets are grouted after installation of the joints. Similar connections can be formed with flat plates or dowels to form hinges (shear connections).

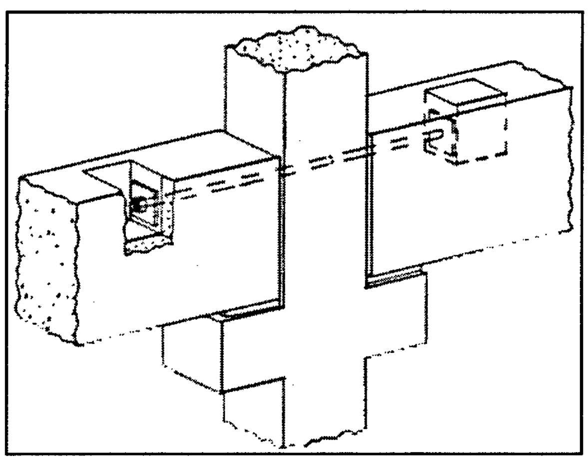


Figure 6.4. Through-bolt connection

A large variety of façade connections have been developed to attach precast panels to support frames. Refer to PCI (1988) for typical details. These connections can be adapted for attachment of precast wall panels or armor panels. Care should be taken to choose corrosion-resistant details and/or to overdesign connections to account for the effects of loss of section.

6-3 Water Stops

Water stops can be classified as rigid or flexible. Suitable rigid water stops are made of copper or stainless steel. These water stops are costly and require special handling to avoid damage. For these reasons, flexible water stops are generally preferred. Flexible water stops can be made of polyvinyl chloride (PVC), butyl, neoprene, and natural rubber. PVC water stops are thermoplastic and can be easily spliced at the jobsite. Suitable compression sealers can be used in lieu of water stops.

Most joint deterioration problems can be traced to failure of water stops. Improperly installed or damaged water stops permit seepage through monolith joints, which in turn accelerates penetration of deleterious agents into concrete around the joints. Water leakage also causes the concrete to become critically saturated and thus susceptible to freeze-thaw damage. Past experience shows that failures of water stops are primarily due to the following reasons: (1) excessive movement of the joints, which rupture the water stops; (2) honeycomb in concrete adjacent to the water stops, due to lack of consolidation; (3) contamination of the water stop surface, which prevents bond to the concrete; (4) puncture of the water stop or complete omission during construction; and (5) breaks in the water stop due to poor or no splice.

Proper selection and installation of water stops is critical to durability of hydraulic structures. In general, water stops must be capable of accommodating the anticipated movements of the adjacent concrete as the joints go through thermal movements. Adequate attention shall be paid in design and construction to allow sufficient consolidation of concrete around water stops. Engineering specification shall require rigorous QC and inspection for installation of water stops. A high degree of workmanship and special attention to splicing, intersection, and supports of water stop are essential. Conventional water stops shall not be used for underwater connections, as discussed in Section 6-6.

6-4 Watertightness Requirements

Watertightness requirements shall be based on project-specific requirements. Joints for permanent floating structures such as floating guard walls shall be watertight. The watertightness of float-in structures is less critical due to their temporary float condition.

Leakage through concrete walls and slabs is an important design consideration if there is a substantial differential hydrostatic pressure on two sides of a concrete wall. In addition, water flow through the joints and cracks could cause long-term durability deterioration, such as leaching, or a reduction in corrosion resistance and freeze-thaw resistance.

In general, a partially cracked concrete section with a compression zone of no less than 30 mm (1.2 in.) is essentially free of leakage. Any section of a watertight component should be designed to have a compression zone no less than 30 mm (1.2 in.) and $0.25h$ in thickness (h is the overall thickness of the

components), whichever is less. For permanently floating structures, the watertight components are generally required to maintain a minimum compressive stress of 0.5 MPa (70 psi) across the section under service loading conditions. The compression should take thermal strains into account. For temporary loading conditions during construction and installation, tensile stresses up to 2 MPa (300 psi) are acceptable. Crack widths are generally checked at critical locations against static loads plus 40 percent dynamic loads.

Attempts to stabilize leaking joints between precast elements have ranged from injecting grout behind or in the joints to the use of hydrophilic compounds. In general, repair of leaking joints in precast construction is similar to repair procedures already developed for cast-in-place construction. To avoid potential leaks, some designers have used tongue-and-groove construction with a grouted void space within the groove. This connection is sensitive to proper cleaning, as discussed below. Typically, grouting is performed using a tremie method, working from the bottom of the joint to the top. In some applications, such as horizontal keyways, voids, or where grout is used to seal a precast element against rock or other nonhomogenous materials, the use of controlled grouting or tube manchete post-grouting may be considered. Both techniques use packers or control valves to limit the region that is exposed to grout pressure, resulting in a series of high-pressure, limited-length grout zones. This may be desirable where a tremie grout would be lost to a void or soft zone.

6-5 Joint Preparation

Precast panel joints generally receive a light sandblast prior to assembly. Other methods such as a moderate pressure wash have been used successfully.

For underwater joints, the exposed P/C mating surface can be severely contaminated with alluvium settlement, which prevents bonding of underwater concrete or grout with the precast surface. Thus, wherever possible, the joint surface should be immediately sealed from flowing water once the segments are lowered to their final positions. Any exposed joint surface shall be cleaned with underwater jetting prior to grouting or concreting.

Vacuum techniques such as air-lift pumps have been developed to clean the inside of drilled shafts and slurry walls. These techniques may be used to clean joints that have collected settlement. Air lifts usually require about 0.33 m of clearance from reinforcing. Air lifts have been used to clean the bottom of shafts to depths exceeding 60 m (180 ft). Commercial divers often have vacuum hose capabilities.

6-6 Underwater Connections

Underwater construction of structural connections usually entails joining two or more P/C segments and filling the voids between the segments with concrete, cement grout, or polymer grout. For isolation joints, the filling materials may be rock, gravel, or polymer foam covered with concrete paving blocks. In essence,

construction of underwater connection entails three essential steps: positioning two or more P/C segments to prescribed tolerances, sealing the joint space between the segments, and grouting or concreting the sealed joint space to make it a monolithic connection. Olmstead P/C wall panels will be anchored by placing tremie concrete behind the panels.

The key to construction of underwater connections is the joining of two P/C sections through underwater grouting or concreting. Since the underwater operations have to be carried out in adverse conditions, with poor visibility and difficult accessibility, construction operations should be carried out above water as much as possible. It is important that the connection detail design facilitates simple and reliable construction. The design of underwater connection shall include or take into account the attainable tolerances, the position-guiding system for mating the adjacent precast segments, sealing of the joints, the performance requirements of grout or concrete mix, the grouting procedure, the grouting system, and the vent system.

Conventional water stops are generally not suitable for construction of underwater connections. To seal a joint, primary considerations should be given to preformed compression seal, Omega seals, Gina seal, or J-seal so as to confine a space in the joint prior to filling the void with suitable materials. Selection of proper material and type of seal depends on the joint design details, installation procedure, and environmental exposure condition.

Strict on-site enforcement of engineering requirements and quality control is paramount for good-quality underwater connection. Under most circumstances, the effectiveness of the divers' inspection is very limited due to the poor visibility. The divers' inspection should be limited to such activities as checking the joint seal and outflow from vents. In principle, on-site monitoring and QC should be carried out mainly above water. Critical items that need careful monitoring include the following: (1) positioning of adjacent P/C segments, (2) sealing of the joints, (3) rate of grouting or concreting placement, (4) grout or concrete delivery system (leakage, plug, or spillover), (5) venting system, and (6) complete grouting of the joints. Field trial testing and post-construction coring shall be considered as a part of the QA/QC program.

In essence, design of underwater connection shall meet the following requirements:

- ***Durability requirements.*** Past experience shows that joint deterioration is the most common durability problem in hydraulic structures. Common causes of joint failure include leakage-induced freeze-thaw damage and corrosion and spalling of monolith joints due to impact and abrasion. Underwater joints are especially vulnerable to physical and chemical attacks. In principle, underwater connections shall be designed for durability against freeze-thaw deterioration, chemical attack, abrasion, erosion, cavitation, spalling due to reactive aggregates, and corrosion of reinforcing steel. Careful design considerations should be given to accommodate the special conditions for underwater construction.

- ***Strength reduction factors.*** Because of the poor visibility and difficult accessibility of underwater work, a higher degree of uncertainty is associated with the quality and integrity of underwater connections than with above-water connections. To take full account of the increased uncertainty in underwater works, structural design of underwater connections shall use a strength-reduction factor of 0.6 for flexure and axial tension and 0.57 for shear and torsion. There is no change in the load factors for underwater connections.

- ***Required characteristics.***
  (1) Construction of underwater connection entails joining P/C modules, sealing the joints from external water, and grouting or concreting the voids with suitable materials. The first requirement of the connection design is to ensure simple and reliable execution of these three steps underwater. To this end, the selection of the seal, the grout mix, the sequence and rate of grouting, and the vent system shall be an essential part of the design.
  (2) All the design forces and moments shall be adequately transferred through the connection by means of shear keyways, reinforcement splicing, and composite action of P/C and underwater grout or concrete.
  (3) Underwater connection shall have durability characteristics at least equivalent to those of above-water connection.

- ***Common details.*** At this writing, most existing connection details were developed for the traditional, in-the-dry method. Precaution shall be taken in using these details for in-the-wet construction, because these connection details may not be suitable or constructible under water or may not have the required characteristics for underwater connections.

- ***Instrumentation.*** For monitoring the construction and performance of underwater connections, instrumentation is recommended.

In general, underwater connections are more costly and more complex to construct than above-water connections. It is often beneficial to minimize the number of underwater connections to avoid an increase in overall project costs.

6-7 Match Casting

Match-cast precast products are typically used in segmental construction to ensure the proper fit-up of mating surfaces between precast segments, while providing the profile grade and horizontal alignment required by design. For hydraulic structures, match-casting applies to horizontal joints (e.g., stacking one element on top of another) and to vertical joints where one element fits tightly against an adjacent element.

Two basic techniques are used to match-cast P/C segments. One technique employs a stationary form, and the other, a form that is moved for every casting. With the stationary form, the first segment is cast with endplates at both ends of

the form. After this segment has been cured to a concrete strength adequate for stripping, it is lifted out of the form and positioned adjacent to the form so that one of its ends serves as the endplate for the match-cast end of the second segment. The other end of the second segment is formed with one of the original endplates.

The positioning of the first segment relative to the form is critical, since it dictates the alignment of the two segments in the completed structure. Sophisticated surveying techniques, together with adjustable screw jacks and stops, are normally used to accurately position the segment. Prior to casting, the match-cast end of this segment is coated with a debonding agent to allow separation of the segments after casting. After the second segment achieves stripping strength, both segments can be stripped from the form. The first segment is moved to storage, while the conventionally formed end of the second segment assumes the role of the endplate for the third segment to be cast.

The "moving form" technique begins in a similar manner. However, after the first segment is cast and cured, it is left stationary on the form pallet. The form is stripped, moved longitudinally, and positioned adjacent to the first segment. The second segment is then match-cast against the first in the same manner as described above. This approach has the advantage of decreasing segment handling, but requires multiple form pallets and significantly more space.

Detailing for connections shall consider the impacts of the detail on the casting operation. Typical precast construction occurs in long casting beds where individual elements are separated by bulkheads. Details that require penetrations of end bulkheads may result in excessive costs associated with casting the elements.

6-8 Bracing

Bracing and guying methods shall be designed to support all construction loads, including wind. Selection of connections shall give due consideration to the size, location, and capacity of the precast panel, bracing, and supporting dead man or anchor. Long span structures may require temporary bracing to adequately support the member and prevent buckling. Precast hydraulic structures can be cast with temporary support walls that are used to brace the element during erection. After final assembly, the support walls are removed.

7 Infill Concrete Placement

7-1 General

Infill concrete placement provides both stability and structural capacity to a number of P/C units used in navigation structures. Infill concrete is used when P/C concrete is used to construct structures such as lock and dam monoliths, unlike structures such as P/C floating approach walls and many types of P/C fixed approach walls that do not require infill concrete. For those P/C structures requiring infill concrete, the infill concrete is needed because without it there would be very little resistance to overturning and sliding. For pile-founded structures, this would result in large numbers of tension piles. For rock-founded conditions, it would result in an unstable structure. In addition, design of the structure would be such that either the panels would become so thick that it would be difficult or impossible to lift or float them into place or the reinforcing required would become excessive.

In addition to the stability and structural capacity that infill concrete provides, consideration must be given to the volumetric changes that occur as a result of using it. Infill concrete goes through a series of volumetric changes due to thermal expansion and contraction and autogenous shrinkage. These volumetric changes can increase the design stresses on the precast elements or can cause the infill concrete to separate from the precast elements, which can result in the P/C and the infill concrete acting independently of one another.

Expansion of the infill concrete due to the heat of hydration of the cement can introduce additional tensile stress to the precast concrete. This additional stress can create extensive cracking in the P/C and excessive stresses in the reinforcement. This expansion can be controlled by using low-heat cements and by placing the infill concrete incrementally. Alternately, increasing the thickness of the precast elements and the amount of reinforcing may be needed. A NISA study, as outlined in ETL 1110-2-365 (HQDA 1994) and discussed in Chapter 1, is helpful in determining such requirements. In addition, ETL 1110-2-542 (HQDA 1997) can be used to perform either a level 1 or level 2 thermal analysis if a more general idea of the thermal effects is needed.

Contraction of the infill concrete can also create problems in the design of precast elements. As the infill concrete cools, it shrinks and tends to pull away from the precast concrete. This shrinkage is in addition to shrinkage that occurs due to the autogenous shrinkage of the concrete. Shrinkage of the infill concrete

creates a problem because, if the P/C and infill concrete are supposed to act compositely, the shrinkage may result in the two concretes acting independently. This problem can be addressed in several ways. One approach is to adjust the concrete mixture and/or placement sequence. Another method is to include studs attached to the precast elements that will provide the load transfer between the precast and infill concretes. Finally, shear transfer can be accomplished by surface roughening of the precast element, as described in Section 8-6. Again, determination of the steps to be taken to mitigate this problem can often be identified through a NISA study. The level 1 and level 2 analyses will not provide the necessary detail needed to determine the effectiveness of load transfer between the infill concrete and the precast concrete.

7-2 Methods

The methodology for performing a NISA study is outlined in ETL 1110-2-365 (HQDA 1994). The guidance document provides information on how the study should be conducted with respect to various modeling parameters, as well as output to be presented. The ETL also provides ranges of material properties for both concrete and foundations so that, if actual properties are not available, the designer can select reasonable values.

Methods also exist for performing simplified thermal analyses based on ETL 1110-2-542 (HQDA 1997). A level 1 analysis as described in ETL 1110-2-542 is simply a one-dimensional thermal analysis from which the strain is calculated; this provides volumetric changes due to a global temperature change of the structure. The level 2 analysis, as described in ETL 1110-2-542, consists of performing a finite element thermal analysis and then computing strains over given portions of the structure based on thermal gradients observed in the finite element analysis.

One of the key items covered in the NISA ETL (HQDA 1994) is to identify when a NISA study should be performed. The ETL requires a NISA to be performed for any one of the following reasons:

a. If the structure is unprecedented.

b. If the structure has exhibited past poor performance.

c. To validate changes in construction parameters for the purpose of reducing construction costs.

Since there is little or no experience with construction of navigation P/C structures within the Corps of Engineers, most of the innovative P/C structures that use infill concrete will require a NISA to be performed. The NISA study may also be used to achieve optimal construction methods. If none of the reasons listed above applies to the structure in question, then level 1 and level 2 analyses may be adequate for evaluating structures.

ETL 1110-2-365 was published in 1994 and is based on NISA studies that were performed on more conventional-type navigation structures that were built using a cofferdam so the structures could be built in the dry. While the ETL was

not written specifically for the analysis of precast shells with infill concrete, it still provides an adequate basis for describing requirements for performing a NISA. It also provides many of the modeling parameters and techniques needed. However, because of the P/C components, some additional modeling techniques will be needed for the modeling of P/C structures built in the wet.

The requirements of a NISA, as described in ETL 1110-2-365 (Appendix A, section A-2), should be followed. This includes the parametric evaluation of material properties, the extreme ambient temperature, parametric evaluation of construction start dates, the duration of the analysis, and the application of service loads. The construction parameters, as described in section A-4 of the ETL, will in general be applicable, except that lift heights may have to be modeled differently and there will not be any insulation for underwater placements. In addition, the P/C structure will need to be modeled as a mature concrete. While the infill concrete will make use of the aging properties capability in ANACAP-U (Anatech Research Corporation 1999), the P/C concrete should be modeled as an elastic material with no aging. However, the P/C concrete should still be modeled using ANACAP-U so that cracking predictions within the P/C concrete can be made. Output from the NISA should be provided as described in section A-7 of ETL 1110-2-365.

7-3 Material Properties

A discussion on materials and mixture design was presented in Chapter 3. The desired characteristics for infill concrete and conventional tests that should be performed were described. Further testing for performing NISA studies should include tests for heat of hydration, creep, autogenous shrinkage, aging modulus of elasticity, and strain capacity. As mentioned above, ETL 1110-2-365 provides a list of the material properties needed when performing a NISA, as well as the expected range of values for each of these material properties. If the exact mixture design is unknown, material properties can often be estimated for preliminary analyses. When estimating material properties, it is best to have a materials engineer who is familiar with the project and the expected mixture design.

To provide a general idea of the material properties of infill concrete, Table 7.1 lists five values of interest for the infill concretes used on the Olmsted Dam and Braddock Dam Projects.

Table 7.1
Material Properties for Infill Concretes

Project	Concrete	Unit Weight kg/m^3 (lb/ft^3)	Thermal Conductivity kW/m-°K (Btu/d-in.-°F)	Elastic Modulus MPa (lb/in.^{2})	Comp. Strength MPa (lb/in.^{2})	Fracture Strain mm/mm-°C (in./in.-°F)
Olmsted	S3-10	2,387 (149)	1.138 (2.283)	21,932 (3.181E6)	7.97 (1156.0)	38.6E-6 (69.5E-6)
	HP2-5	2,387 (149)	1.115 (2.238)	26,028 (3.775E6)	12.60 (1827.5)	38.1E-6 (68.5E-6)
Braddock	C1-7 (Wet)	2,390 (148.6)	1.246 (2.50)	17,664 (2.562E6)	20.03 (2905)	122.2E-6 (220E-6)
	C2-4 (Dry)	2,326 (145.2)	1.171 (2.35)	11,183 (1.622E6)	10.00 (1451)	94.4E-6 (170E-6)

8 Construction Details

Notation

ASD = allowable strength design

8-1 General

This document considers the construction of thin-walled concrete structures and structural segments that have been prefabricated into sufficiently large sections that they require water transport to the installation site. For short-term installation forces that are experienced during load-out, transportation, and installation, the allowable stress limits in the elements designed according to ASD requirements may be increased by one-third.

Any buoyant compartments within a segment shall be designed to resist hydrostatic pressures on the compartments for all stages of construction, including load-out, intermediate staging, transport, and installation.

A construction and installation manual describing all critical aspects of construction, transport, and installation should be prepared.

a. ***Construction stages.***

(1) *First-stage prefabrication.* The initial construction work should be performed away from the permanent site in a protected offsite area at, or near, shore, to the maximum extent practicable. Examples of initial prefabrication facilities include skid-ways, graving docks, dry docks, shipyards, finger piers, moles, barges, locks, and fabrication-yards.

(2) *Load-out.* Load-out of the prefabricated segments into the water from the offsite prefabrication facility can be achieved by a variety of means including skidding, launching, float-out, lift-out, sinking, and lowering. The load-out forces should be determined for combined static and dynamic forces, considering the action of the structure itself, applied dynamic forces, and environmental forces including wind, waves, and current present during the load-out operation.

(3) *Intermediate-stage prefabrication and outfitting.* The segments may be incomplete at the time of load-out from the prefabrication facility.

If so, further stages of construction, and/or outfitting, with the structure afloat, or temporarily grounded, in a protected location may be necessary prior to final transport and installation. Calculations shall be made of stress changes during intermediate construction to avoid overloading any segment components. If the segment is temporarily grounded, the bottom should be prepared to acceptable tolerances to avoid overstressing the segment.

(4) *Transport.* All aspects of the transportation phase of work should be designed and planned to ensure that the segment components and towing points (bollards, towing brackets, collars, yokes, etc.) are not overstressed.

Some structures (e.g., floating approach walls) may be sensitive to fatigue. If so, the fatigue accumulated during transport should be considered together with the service fatigue when assessing a structure's fatigue life.

The motion response of the segment and any supplemental buoyancy tanks and/or lifting vessels should be determined in the transport condition. Checks should be made for the extreme motion conditions to ensure that adequate draft is provided. The effect of structural accelerations during transport should also be evaluated.

If an air cushion is used beneath the segment to reduce draft during early construction, or transport, the effects of bending forces and accidental loss of air should be considered. Adequate equipment and instrumentation shall be provided to adequately regulate and monitor the air pressure and water level in any such compartments.

For a towed segment, an adequate number of towing connections and towing lines shall be provided and appropriately positioned. Attachments and towing lines shall be designed with a minimum load factor of 2, unless otherwise indicated.

Compartmentation and damage control should be considered to ensure adequate stability and buoyancy in the event of accidental flooding.

(5) *Installation.* All aspects of the installation of the segments, including sinking/lowering, positioning/station-keeping, and setdown, should be thoroughly planned. If required, ballast lines should be provided and procedures developed for the sinking operations. Adequate guides and/or station-keeping capacity should be provided to ensure that the segment is positioned to within the tolerances specified. Global positioning survey devices have been used successfully to position large precast structures during transport and installation. Adequate foundation capacity and adjustability shall be provided during setdown operations.

Fendering may be required to avoid impact with adjoining structures, and surveys should be performed before, during, and after segment installation.

(6) *Final in situ construction.* Once the segment is installed, any in situ construction should be performed to complete the structure. This work may include placement of tremie concrete, underbase grouting, and infill with granular materials like sand, gravel, or stone. In situ construction may also include in-the-dry construction on top of the installed segment and/or the installation of gates, mechanical equipment, and other appurtenant features.

b. ***Buoyancy and floating stability.***

(1) *Tolerances and control.* Tolerances and weight control can have an important impact on the buoyancy and floating stability of a segment (see Appendix C). In setting these tolerances, attention should be given to the following factors that can affect the center of gravity, draft, and metacenter of the segment:

- Unit weight of the concrete without reinforcing.
- Weight and distribution of reinforcing steel and appurtenances.
- Absorption/loss of water by the concrete with time.
- Accuracy of dimensions of member sizes.
- Control of overall geometry of the segment.
- Weights and distribution of all equipment, attachments, construction aids, or buoyancy tanks.
- All solid and liquid ballast, including control of effective free water planes inside the structure.
- Specific gravity of the water including variations due to sediment and salts.

(2) *Supplemental buoyancy.* Buoyancy tanks with atmospheric internal pressures should be designed to resist the maximum credible external pressures for all stages of construction, including accidental cases. If necessary, provisions can be made to pressurize the buoyancy tanks to avoid collapse. Buoyancy tanks must be connected to the segment with sufficient strength and rigidity so as to remain at the proper attitude, to resist cyclic fatigue, and to withstand accidental impacts during construction. Release of the temporary tanks should be carefully planned.

(3) *Solid ballast.* Solid ballast in the form of sand, stone, or iron ore may be used to improve floating stability, attitude, and draft. Solid ballast can also improve the segment stability when grounded.

c. ***Fabrication and installation loads.***

(1) *General.* The strength requirements given in Chapter 10 give guidance for both basic load factors and strength reduction factors, and some guidance for load combinations for fabrication, transportation, installation, and operation load cases. The purpose of this section is to expand on the requirements (given in Chapter 10) for ensuring that the structure begins its service life with its designed strength and structural integrity intact.

(2) *Dynamic effects.* Static loads should be increased by appropriate dynamic load effects to determine the nominal loads for the design of elements affected by dynamic fabrication and installation load cases. These nominal loads can be determined by a suitable dynamic analysis, or they can be based on previous measured experiences. For lifts made onshore or in sheltered waters, the minimum load factor for dynamic effects shall be 1.15. However, Table 5.4 presents alternate dynamic effect factors that can be used for well-controlled conditions.

(3) *Load factors.* The nominal loads should be further increased by appropriate load factors to determine the internal member forces to be used for strength checks. For equilibrium purposes, it may not be possible to apply different load factors for each of the gravitational, environmental, buoyancy, and inertial loads for all stages of fabrication and installation. For such cases, load factors of 1.0 should be applied to all external loads, and a factor of 1.4 should be applied to the nominal internal member forces. For cases where external loads can be factored independently before the loads are combined, see Section 10-1. For lifting cases with form stripping loads, see Table 5.1. For cases of bottom-founded gravity structures (without piles), to determine global overturning stability, the gravitational loads should use a 0.9 load factor instead of the 1.4 factor used for strength design.

(4) *Local effects for lifts.* Due to variations in load distribution around lifting points, a further local effects factor should be applied as follows:
For lifting eyes and spreader beams..1.33
For other structural members transferring lifting forces................1.15

(5) *Side loading for lifts.* Lifting eyes and connections to the supporting structural members should be designed for a horizontal force of 5 percent of the static sling load, applied simultaneously with the sling loads. This load should be applied perpendicularly to the center of the lifting eye or connection.

(6) *Effects of tolerances for lifts.* The lifting factors applied in this section assume the fabrication tolerances cited in Appendix C and that variations in the sling lengths do not exceed ±0.2 percent of the nominal sling length, or 38 mm (1.5 in.). The total variation from the longest to the shortest sling should not be greater than 0.5 percent of the sling length, or 75 mm (3.0 in.). If either the fabrication tolerances or the sling length tolerances exceed these limits, a detailed analysis could be made to determine the effects of these variations.

(7) *Slings, shackles, and fittings.* For normal inland waterways lifting conditions, slings, shackles, and fitting should be selected to have a factor of safety of 4.0 for the manufacturer's rated minimum breaking strength of the rigging compared to the nominal, or static, load. This factor of safety should be increased for severe conditions, and may be reduced to a minimum of 3.0 for carefully controlled conditions.

8-2 Embedments and Penetrations

Embedments

Local stress concentrations and local interferences with reinforcing steel caused by embedments should be carefully evaluated and detailed. It may be necessary to supplement and/or relocate reinforcing steel in the vicinity of embedments. The influence of load factors on embedments used for lifting, handling, and transport should be considered.

Penetrations

Penetrations through concrete members, such as piping, cables, and mechanical devices, should be carefully evaluated with regard to local stress concentrations and interferences with reinforcing steel. It may be necessary to supplement and/or relocate reinforcing steel in the vicinity of penetrations.

Sacrificial anodes

Sacrificial anodes may be required to protect embedments and penetrations from corrosion.

8-3 Construction Joints and Concrete Placement

Joint preparation

Construction joints should be prepared with extra care when a structure is to be permanently or temporarily watertight. Appropriate measures to provide watertight joints include removal of laitance by water-jetting or sandblasting, use of epoxy primers, and use of cementitious primers.

Concrete placement

Hot and cold weather placement. Care should be taken when placing concrete in either hot or cold weather conditions. ACI 306R and ACI 305R provide guidance on these topics. Special attention to cold weather practice is required to prevent damage to open ducts, which may contain water subject to freezing.

8-4 Detailing Requirements for Specialty Reinforcement

Headed reinforcement

Headed reinforcement, as defined by ASTM A 970, can be used to relieve congestion of reinforcing steel, to develop reinforcing bars in a shorter length, to control load paths into the reinforcing steel, to allow for the use of larger reinforcing bars, to provide better confinement to the concrete, and to facilitate construction. The head is provided on the bars to anchor the reinforcement to the concrete without the need to rely on bond. The heads are available in a variety of shapes (square, rectangular, round, oval) and sizes (No. 5 bar to No. 18 bar). ACI 349 provides guidelines for the design of mechanical anchorages. Care shall be taken to ensure sufficient room for multiple or interconnected headed reinforcing. Some close spacing or bundling of the reinforcing may not be possible for headed bars, with large head dimensions.

Mechanical connections

Mechanical couplers may be used in tension or compression provided they provide 125 percent of the specified yield strength of the reinforcing steel bars. Available techniques include swaging, threaded, sleeves with fill, and clamped. Further guidance can be found in Section 3-5 of this report and in ACI 318R.

8-5 Detailing Requirements for Prestressing Reinforcement

Prestressing tendons shall be confined within the reinforcing steel stirrups in webs and between layers of transverse reinforcing steel in slabs, walls, and flanges.

Curved prestressing tendons shall be adequately confined by lateral reinforcement. Spacing of the confinement reinforcement shall not exceed either 3 times the outside diameter of the duct or 600 mm (24 in.).

When the tendons curve away from the longitudinal member, such as at a typical intermediate anchorage, prestressing force will produce transverse forces that are radial to the tendons. The transverse in-plane force tends to shear off the concrete cover and split the concrete at the junction. The transverse deviation force must be calculated, and fully anchored tieback stirrups must be provided accordingly to resist the transverse force.

When post-tensioning ducts are spaced closer than 300 mm (12 in.) in slabs, the top and bottom reinforcement mats should be tied together with No. 10 hairpin bars. The spacing between the hairpin bars shall not exceed 450 mm (18 in.) or 1.5 times the slab thickness.

The clear spacing between straight post-tension ducts shall not be less than 38 mm (1.5 in.) or 1.5 times the maximum size of the coarse aggregates. The clear horizontal spacing between post-tensioning duct bundles shall not be less than 100 mm (4 in.). The clear vertical spacing between the duct bundles shall not be less than 38 mm (1.5 in.) or 1.5 times the maximum size of the coarse aggregates.

8-6 Surface Treatments for Composite Action

When precast concrete shell structures are designed to behave compositely with cast-in-place or tremie concrete, shear must be transferred across the interface between the two concrete layers. The two separate concrete placements are intended to act as a unit when resisting externally applied loads. The surface treatment of the precast component influences the mechanism of shear transfer across the interface with tremie concrete. Surfaces can be smooth formed, sandblasted, intentionally roughened, corrugated, or patterned with shear blocks or holes. Corrugated surfaces with sufficient amplitude develop the shear capacity of concrete.

Typical designs use the "shear friction" concept at the interface. Design advantages are realized when the surface of the precast member, which will interface with cast-in-place concrete, is intentionally roughened to full amplitude of approximately 1/4 in. Although the shear friction concept does not require roughening, a roughed surface has approximately 40 percent more capacity than a smooth surface. Roughening of surfaces is very common in the precast industry. Methods used depend upon whether the surface to be roughened is exposed or formed.

A requirement common to both exposed and formed roughened surfaces is that they must be clean and free of laitance prior to placing the cast-in-place concrete. It is also generally desirable to moisten the precast surface prior to the second placement.

The standard method of roughening exposed surfaces is to "rake" or "broom" the concrete while it is still in its plastic state. After the concrete has been struck level, a workman rakes the surface with a tool that creates grooves at a specified spacing and depth. These grooves normally run transverse to the direction of the anticipated shear force and must be deep enough to produce the desired roughness, but not so deep so as to dislodge individual aggregate particles near the surface.

Formed surfaces cannot be roughened in the same manner as exposed surfaces. Several methods used to roughen formed surfaces include chemical surface set retarders, deep sandblasting or shot blasting, castellations, shear keys, and corrugated surface.

Surface set retarders, which locally retard the setting of cement, are painted onto the form in the desired location prior to casting the concrete. After form removal, the retarder is pressure washed from the concrete surface, resulting in a roughened, exposed-aggregate finish. Set retarders are formulated with different

strengths to result in varying depths of retardation. Normally, the strongest formulation is required to achieve the roughness desired for composite action. Sandblasting and shot blasting are done manually after the product is stripped. They are labor intensive. Shear keys and castellations are formed into the concrete surface. Roughened formed surfaces are normally used at the interface with cast-in-place concrete joints.

Bush hammering is not recommended since it may actually degrade the bond strength at the composite interface.

8-7 Detailing at Openings and Penetrations

Openings and penetrations are frequently required through thin-wall structural sections. Such openings and penetrations necessitate a number of detailing refinements to the basic design of the thin-wall section including

a. ***Openings/penetrations without significant stress concentrations.*** This detailing category includes common cases, such as normal pipe penetrations and steel embeds. For such cases, the strength of the penetration inhibits significant stress concentration; however, the penetration may interrupt the base steel reinforcing layout. For cases when minor adjustments in the layout of the reinforcing steel cannot accommodate the penetration, it is common practice to terminate the reinforcing bars disrupted by the penetration and to provide an area of supplemental reinforcing bars equal to that of the terminated bars, at the edge of the penetration, with the same orientation as the terminated bars.

b. ***Openings/penetrations with membrane stress concentrations.*** This detailing category includes access openings, lightening openings, and empty prestressing ducts. For such cases, the extent of the concentration of the stress fields in the concrete member is a function of the opening/penetration (e.g., shape, size, and thickness); the member itself (e.g., shape, size, material, thickness); and the stress fields (e.g., compression, tension, or shear) in the member. Calculation of the stress concentrations is typically determined either from tabulated classical formula or from refined localized finite element method models. Efforts should be made to minimize the membrane stress concentration by rounding, or chamfering, acute angles of the openings/penetrations, or by supplemental strengthening of the openings/penetration using such methods as flanges around the edges of the opening/penetration, temporary structural covers/diaphragms, or infill materials. Once the final stress concentration is determined, sufficient supplemental steel reinforcing is required to be added around the opening/penetration.

c. ***Openings/penetrations with concentrated punching shear stress.*** This detailing category includes cases such as openings/penetrations near concentrated external punching shear loads (such as vessel impact, ice loading, etc.) and concentrated internal punching shear loads (such as columns, landing/support points, etc.). When such concentrated punching

shear loads are located within 5 times the effective depth of the thin-wall structural member, the punching shear capacity of the thin-wall member shall be reduced accordingly.

8-8 Detailing for Preventing Delamination

Delamination cracking occurs, in the longitudinal direction, parallel to the main reinforcing of a concrete member and thus frequently requires supplemental transverse reinforcing steel to control. As described in the subparagraphs below, delamination cracks occur for a variety of reasons, including changes in the direction of prestressing tendons, stresses (either active or residual) from thermal gradients (either current or past), and the linking together of splitting cracks aligned along the axis of the member. Supplemental reinforcing may be required for any other case where the splitting tensile forces exceed the tensile strength of the concrete so as to create continuous longitudinal cracking.

a. ***Delamination from changes in the direction of prestressing tendons.*** When prestressing tendons deviate from a straight line, radial/transverse stresses are developed between the tendon and the surrounding concrete to maintain force equilibrium. These radial/transverse stresses can cause delamination cracking unless suitable reinforcing steel is provided. Wobble of a prestressing duct, or cases where construction workers bend a duct around an obstruction, can also cause a tendon that was intended to be straight to generate unplanned transverse stresses in the adjoining concrete, which can either cause delamination cracks within the core of a concrete member or spalling of the concrete cover. Another frequent example of such potential delamination occurs where prestressing tendons change direction to exit a member to an intermediate anchorage. Furthermore, tendons following the mid-thickness of a curved concrete member, such as a dome or a cylindrical tank, also induce radial stresses, which may require additional radial reinforcing if the tensile strength of the concrete is exceeded.

b. ***Delamination due to thermal gradients.*** Thermal gradients can induce radial/transverse stresses in the concrete that may cause delamination cracking, particularly in early-age concrete where the tensile strength of the concrete is low, and the heat of hydration of the concrete can cause thermal gradients. Typical solutions to such potential problems include using low heat of hydration cement, precooling the fresh concrete, insulating the surface of the concrete, and providing supplemental temperature steel.

c. ***Delamination due to linked longitudinal cracks.*** Closely spaced stress concentration points, such as closely spaced empty prestressing ducts, subject to sufficient orthogonal stresses, can introduce splitting tension cracks that can link together to form delamination cracks. Typical solutions include increasing the spacing between the stress concentration points, decreasing the orthogonal stresses whose deflection around the

stress concentration points causes the splitting tension stresses, and providing supplemental reinforcing to resist the splitting tension stresses.

9 Serviceability Requirements

Notation

A = cross-sectional area of concrete surrounding each steel bar
A_s = area of nonprestressed reinforcement
A_p = area of prestressed steel
C_c = thickness of concrete cover
C_1 = initial multiplier for long-term deflection calculation
C_2 = revised multiplier for long-term deflection
d_c = distance from extreme compression fiber to centroid of tension reinforcement (mm)
f_s = stresses in steel reinforcement
f_d = stress due to unfactored dead load at extreme fiber of precompressed tension zone
f_{pe} = concrete compressive stress due to effective prestress after losses at the extreme fibers of the precompressed tension zone. (For nonprestressed P/C members, f_{pe} equals zero)
Δf_{ps} = maximum stress increase in prestressing steel
I_e = effective moment of inertia accounting for effects of cracking and reinforcement
I_g = moment of inertia of gross cross section, ignoring reinforcement
I_{cr} = moment of inertia of cracked section transformed to concrete
M_{cr} = calculated in accordance with Section 9.4
M_a = unfactored maximum moment in member
W = crack width, in units of 0.03 mm
y_t = distance from centroidal axis to extreme fibers in tension
λ = multiplier for the short-term deflection calculation
β = ratio of distance to the neutral axis from the extreme tension fiber to distance from the centroid of the main reinforcement and prestressing steel

ρ' = reinforcement ratio of the nonprestressed compression reinforcing steel

ξ = time-dependent factor for sustained loads in calculation of long-term deflection

The serviceability requirements include satisfactory performance and durability of the structural member during the design life of the hydraulic structures. This chapter on strength and serviceability requirements of precast/prestressed concrete expands the guidance in EM 1110-2-2104.

9-1 Strength and Serviceability Requirements

All P/C members of hydraulic structures must satisfy serviceability requirements. The serviceability requirements include satisfactory performance and durability of the structural member during the design life of the hydraulic structures, usually including three areas of structural behavior: durability performance, crack width, and deflection.

In general, the crack width and deflection requirements are checked by engineering calculations with the Working Stress Design method. Other durability requirements shall be met by means of proper structural detailing and construction QC.

9-2 Durability Requirements

a. *Concrete quality.* The long-term durability performance of hydraulic structures is largely due to the in situ concrete quality. To achieve durable concrete for hydraulic structures, the water-cementitious materials ratio and the minimum compressive strength of concrete shall meet the requirements in Table 9.1.

Table 9.1
Concrete Quality Requirements

Exposure Conditions	Area Subjected to Abrasion	Other
Maximum w/c ratio	0.40	0.45
Minimum total cementitious materials	380 kg/m^3 (640 lb/yd^3)	340 kg/m^3 (574 lb/yd^3)
Minimum compressive strength at 28 days	40 MPa (5800 psi)	30 MPa (4300 psi)

The minimum concrete quality requirements themselves, presented in Table 9.1, do not guarantee high-quality in situ concrete. The concrete mixture design and concrete production procedures must be conducted to consistently achieve the required strength, workability, setting time, density, permeability, and durability characteristics.

b. *Minimum concrete cover.* Minimum concrete cover shall vary with the applications and exposure conditions of P/C components. The minimum concrete cover to the main reinforcement parallel to concrete surface shall follow the requirements specified in Table 9.2. The minimum cover for stirrups and ties shall be 15 mm (0.6 in.) less than the cover specified in Table 9.2. These requirements supersede the cover requirements in EM 1110-2-2104 (HQDA 1992).

Table 9.2
Concrete Cover Requirements

Application	Minimum Concrete Cover
Surfaces exposed to cavitation or severe abrasion and erosion.	100 mm (4 in.)
Surface exposed to moderate abrasion and erosion or where water velocity exceeds 8 fps	75 mm (3 in.)
Surface exposed to earth, weather, or where water velocity less than or equal to 8 fps	50 mm (2 in.)
Surface adjacent to structural infill placements	One bar diameter for No. 14 or larger; 1 in. for No. 11 bars and smaller

c. *Resistance to freezing and thawing.* Where concrete is exposed to a repeated freezing condition during the operation phase of the structure, air-entraining admixture shall be used in the concrete. Before concreting operations commence, tests shall be conducted to determine the air void system in concrete produced with the proposed materials, mixing, delivery, and placing systems. A minimum surface area of the air voids of 25 mm^2/mm^3 and a maximum spacing factor of 0.20 mm shall be produced when the concrete is tested in accordance with ASTM C 457. The level of air content, in fresh concrete, necessary to meet these air void parameters shall be used to establish the required air content for the subsequent jobsite testing. All ducts, rod holes, or other small openings in the concrete shall be filled, unless alternative means of excluding water that may freeze have been provided. Unfilled holes in concrete may later fill with water and freeze, causing significant bursting forces that may severely crack the concrete.

d. *Abrasion resistance.* In regions of hydraulic structures where severe abrasion due to gravel, sand, silt, ice, or barge is expected, the concrete compressive strength and concrete cover shall meet the minimum requirements in Tables 9-1 and 9-2.

e. *Bar supports and ties.* Bar supports shall be nonmetallic or epoxy-coated, unless the supports are protected by a concrete cover that meets the requirements in Table 9.2. Epoxy-coated reinforcement shall be fastened with epoxy-coated or plastic-coated wires.

9-3 Crack Control

All precast/prestressed concrete components of hydraulic structures shall be classified into three categories: no cracking, controlled cracking, and no

requirement on crack width. The classification shall be specified in the project design criteria in accordance with the serviceability requirements of each precast component and in consultation with Headquarters, USACE.

The purpose of limiting crack widths is to achieve the following objectives:

- Acceptable level of corrosion resistance.
- Durability by limiting ingress of aggressive agents and minimizing freeze-thaw exposure.
- Intended structural behavior, e.g., transfer of membrane shear.
- Watertightness.
- Special project requirements, such as abrasion or impact resistance.

In relation to the durability of reinforced concrete structures against corrosion attack and other deleterious actions, crack control has been subjected to extensive research in the past. Among the extensive studies are the U.S. Army Engineer Waterways Experiment Station's (WES) long-term exposure tests (Roshore 1971), the U.S. Navy's concrete durability study (Hayes 1980), the British Department of Energy's "Concrete in the Ocean" program (Wilkins and Lawrence 1980), and Norwegian offshore and coastal field surveys of marine structures (Gjorv 1968, 1996).

These researchers have reached a number of conclusions that can be used as a basis for engineering design. In general, cracks will lead to initiation of corrosion by allowing deleterious materials (salts, water, oxygen, etc.) to easily penetrate into the interior of concrete. The corrosion, in turn, promotes further cracking of the concrete through the mechanism of metal expansion. Although the crack width does influence the time to initiation of the corrosion, extensive research indicates that the crack width is of secondary importance to the rate of corrosion in comparison with concrete quality and concrete cover (Wilkins and Lawrence 1980, Mehta and Gerwick 1982, Beeby 1983, Brook and Stillwell 1983, Gautefall 1983). It is generally not the crack width but, rather, the surface area covered by the cracks that has significant effect on serious corrosion damage. Thicker concrete cover over reinforcement will result in greater surface crack width. However, these wider surface cracks are not detrimental to the corrosion protection of the reinforcement.

Leakage through concrete cracks is an important design consideration if there is a differential hydrostatic pressure on two sides of a concrete wall. Extensive through thickness cracks must be avoided. But minor leakage has historically been acceptable for some structures. A partially cracked concrete section with a compression zone of no less than 30 mm (1.2 in.) is essentially free of leakage. Consequently, some design criteria (ACI 357R) require that any section of a watertight component be designed to have a compression zone no less than 30 mm and $0.25h$ in thickness (where h is the overall thickness of the components).

Transverse cracking of concrete often plays an important role in fatigue design of reinforced concrete structures. The current design philosophy is to minimize the possibility of such damage by limiting crack occurrence. For critical members subjected to repeated loading, fatigue design criteria commonly

require that no membrane tension be allowed and that flexural tensile stresses be limited below the modulus of rupture.

In principle, the crack control criteria for concrete structures should be based on at least four important factors:

- Performance criteria and environmental exposure of the structure.
- Concrete cover to reinforcement.
- Concrete quality.
- Minimum reinforcement to limit crack width.

When applying a general design code, such as ACI 318, to design hydraulic concrete structures, there will generally be some disparities between the code provisions and the performance requirements of the structures. Therefore, substantial experience and engineering will be required in specifying the crack criteria for hydraulic structures due to their special requirements and environmental conditions.

Crack width of concrete is inherently subject to wide scatter, even in carefully controlled laboratory tests, and is influenced by shrinkage and other time-dependent effects. In principle, the best crack control is obtained when the steel bars are well distributed over the tension zone of the concrete. Several bars at moderate spacing are more effective in controlling cracking than fewer large bars of equivalent area.

Research shows that the crack width of concrete under service conditions is proportional to steel stress. However, the thickness of concrete cover and the area of concrete in the zone of maximum tension surrounding each individual reinforcing bar are the other two significant factors. Therefore, the concrete cracking is closely related to steel stresses, concrete cover, and steel bar spacing.

At present, crack control criteria in most design codes and specifications can be classified into five categories:

- Allowable stress design (ACI 357, API).
- *Z*-factor method (AASHTO LRFD, ACI 318-95).
- Hydraulic factor design (EM 1110-2-2104).
- Allowable maximum bar spacing (ACI 318-99).
- Crack width criteria.

(1) *Allowable stress criteria*: The crack control criteria are established on the basis of limitations of the steel stresses. The criteria stipulate that concrete crack control is satisfactory, if the maximum stresses in reinforcement f_s and the maximum stress increase in prestressing steel Δf_{ps} meet the limitations in Table 9.3.

Past experience shows that, under most circumstances, the allowable stress design criteria not only provide effective crack control, but also simplify the design. The adequacy of these design criteria has been demonstrated by the excellent service performance of many structures

that have been designed in accordance with the criteria over the last several decades.

Table 9.3 Allowable Stress Design Limitation for Crack Control	
Loading Condition	**Change in Steel Stess**
At the stages of construction, transportation, installation, and inspection	$\Delta f_{ps} \le 127.6 MPa\ (18.5 ksi)$ $f_s \le 158.6 MPa\ (23 ksi)$
Under normal service condition	$\Delta f_{ps} \le 75.8 MPa\ (11 ksi)$ $f_s \le 117.2 MPa\ (17 ksi)$
Under extreme loading at service	$f_s \le 0.6 f_y$

(2) *Z-factor method*: AASHTO Load-and Resistance Factor Design specifications adopt the crack control design method. ACI 318-95 has similar provisions for crack control. The method requires that the tensile stress (f_{sa}) in reinforcing steel bars under service conditions be limited by the following value:

$$f_{sa} = \frac{Z}{(d_c A)^{1/3}} \le 0.6 f_y \tag{9-1}$$

where

Z = 30,000 N/mm (170 kips/in.) for moderate exposure conditions
= 23,000 N/mm (130 kips/in.) for severe exposure conditions
= 17,500 N/mm (100 kips/in.) for buried structures
d_c = 50 mm (2 in.), if the concrete cover exceeds 50 mm (2 in.)
A = cross-sectional area of concrete surrounding each steel bar. [For calculation, the thickness of clear concrete cover used to compute A should not exceed 50 mm (2 in.).]

The criteria are essentially based upon the allowable stress criteria and the Gergely-Lutz equation. For different exposure conditions, there are different Z-factors corresponding to different allowable crack widths.

In addition to the above Z-factor criteria, AASHTO LRFD Specifications provide further guidance for crack control design:

> Structures subject to very aggressive exposure are beyond the scope of these provisions. For such conditions, more restrictive limits on crack widths may be required. Narrower surface crack widths may be obtained by using the recommendations in ACI 350R.

The ACI 350 method is essentially the same as the hydraulic factor design in EM 1110-2-2104.

(3) *Hydraulic factor design*: Engineer Manual 1110-2-2104 stipulates that the factored load combinations under service conditions for design load, as prescribed in ACI 318, shall be increased by the hydraulic factor H_f. Factor H_f equals 1.3 for all loads except for direct tension. For members in direct tension, H_f equals 1.65. The use of the hydraulic factor in the ultimate strength design method is intended to limit the stresses in reinforcing bars to approximately the same level as those designed according to the Working Stress Method. Therefore, the hydraulic factor design method can be perceived as a simple method to control the cracking of concrete under service condition by limiting steel stresses.

Design Example 3 (Appendix E) shows that the steel stresses in a typical precast concrete thin-wall member, as designed in accordance with EM 1110-2-2104, often fall within the range of 20 to 25 ksi (140 to 175 MPa).

For in-the-wet construction, the primary loads on P/C elements typically occur during construction stages (transport and installation). Once installed in their final positions, these elements are normally used as in situ form for massive tremie concrete and act compositely with the tremie concrete under service conditions. Therefore, the hydraulic factor method is not directly applicable for the most critical load combinations (during construction) on the precast elements. For construction loads, the allowable stress criteria are appropriate for control of cracking in these precast elements.

(4) *Allowable bar spacing method*: This design method is adopted in the latest edition of ACI 318-99 to replace the Z factor requirement of previous editions. The requirement is as follows:

$$s = \frac{540}{f_s} - 2.5C_c \tag{9-2}$$

where f_s is calculated stress (in kips per square inch) in reinforcement at service load, and C_c is thickness of concrete cover (in inches). Variable f_s can be computed by dividing the unfactored moment by the product of steel area and internal moment arm. Alternatively, f_s may be taken as 60 percent of specified yield strength of reinforcement.

The required bar spacing for crack control is independent of the exposure condition. This leads to significant difference in spacing between the new spacing requirement and the previous requirements (ACI 318-95) for structure exposed to severe environmental conditions. The ACI 318 Committee provided a number of reasons for the recent change in crack control design method:

- To criticize the previous crack control method (*Z*-factor method in ACI 318-95), the committee believes that steel stress is the most important parameter in crack control. Most of the correlation between test results and crack width prediction formulas is correlated with steel stress. When the steel stress is held constant, there is very little correlation with

the value $(d_c A)^{1/3}$. Although the Gergely-Lutz regression line is drawn through the scattered data, any other line would fit almost as well.

- It is misleading to purport to calculate crack widths, given the inherent variability in cracking. The committee prefers to specify bar spacing directly as a function of steel stress and concrete cover, without implying a calculated crack width.

- The role of cracks in the corrosion of reinforcement is controversial. Researchers show that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. Concrete quality, adequate compaction, and ample concrete cover may be of greater importance for corrosion protection than crack width at the concrete surface. For this reason, the distinction between interior and exterior exposure in the previous editions of ACI 318 has been eliminated in ACI 318-99.

In addition, ACI 318 provides further guidance for crack control design:

> The allowable bar spacing requirements "are not sufficient for structures subject to very aggressive exposure or designed to be watertight. For such structures, special investigations and precautions are required."

Past experience indicates that crack control is highly dependent upon the exposure conditions and performance requirements of the structures. More strict crack control is necessary for hydraulic structures subject to severe exposure conditions. Abrasion resistance, freeze-thaw resistance, and watertightness, as well as corrosion control, are several design considerations in crack control. As stated in ACI 318 Commentary, "the new provisions for spacing are intended to control surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure."

Comparisons have been made between the new crack control provisions (ACI 318-99) and the previous *Z* factor method (ACI 318-95) for varying thickness of concrete cover and different exposure conditions (Figure 9.1). The review indicates that the new provisions significantly relax the spacing requirements for concrete cover between 50 mm (2 in.) and 100 mm (4 in.). For example, the *Z*-factor method requires 50-mm (2-in.) reinforcement spacing for 100-mm (4-in.) concrete cover, while the new ACI provisions require approximately 150-mm (6-in.) reinforcement spacing.

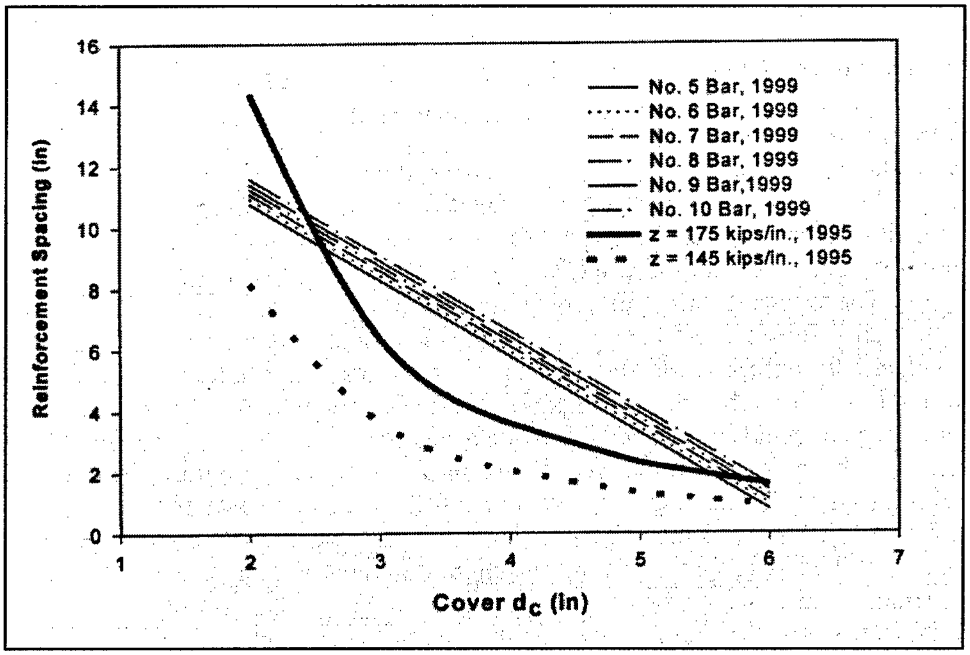


Figure 9.1. Comparison of bar spacing for slabs per ACI 318-99 and ACI 318-95 for varying thickness of concrete cover and exposure conditions

In conclusion, the new ACI crack control provisions appear to work well for limiting surface cracks for purpose of appearance. But it is not strictly applicable to hydraulic concrete structures when durability, serviceability, and watertightness are important.

(5) *Crack width criteria*: The allowable crack width criteria are almost exclusively used in the serviceability check. This trend has been supported by sophisticated theories for crack width analysis and by sophisticated computer analysis. Analysis and design for serviceability limit state (SLS), particularly the crack width control requirement, is often performed by a "point-to-point" check. Sectional forces from every single element of a global finite element analysis have to be analyzed to document the SLS.

In the authors' opinion, the recent trend toward more complex and stringent crack control criteria has not always resulted in cost-effective design. The "point-to-point" check often leads to problems with unrealistic peak stress values, such as stress concentration at corners. The computer analyses often pick up details of nominal infringement of design criteria that has no significant consequence to the structural safety, serviceability, and durability. In many cases, the amount of reinforcement required for crack control is 15 to 30 percent more than needed to satisfy the ultimate limit state requirements. It is questionable whether crack width design criteria with emphasis on a difference as little as 0.05 mm are a valid measure of corrosion susceptibility.

For hydraulic surfaces that are exposed to flowing water, crack width at the concrete surface shall be imposed to enhance the abrasion/erosion resistance and

leakage control. The crack width criteria are based on the allowable values in Tables 9-4 and 9-5 for sustained loading and temporary loading, respectively.

Calculation of the crack width (W, in units of 0.03 mm) shall follow Provision 10.6 of ACI 318 as follows:

$$W = 0.076\, \beta f_s \sqrt[3]{d_c A} \tag{9-3}$$

where β may be approximately taken as 1.2 for beams and 1.35 for plates and slabs and

f_s = working stress in reinforcement, MPa
A = effective tension area of concrete surrounding the flexural tension reinforcement and having the same centroid as that reinforcement, divided by the number of bars or wires, mm

Figure 9.2 illustrates the design parameters d_c and A.

Table 9.4
Allowable Crack Width Under Sustained Loading

For nonprestressed concrete components	
Surfaces subjected to abrasion/erosion action of flowing water	$W \le 0.15\,mm \quad (0.006\,in)$
All other exterior surfaces	$W \le 0.33\,mm \quad (0.013\,in)$
For prestressed concrete components	
Surface subjected to abrasion/erosion action	$W = 0$
All other exterior surfaces	$W \le 0.20\,mm \quad (0.008\,in)$

Table 9.5
Allowable Crack Width Under Temporary Loading (Construction, Transportation, and Inspection)

Type	Crack Width
For through-thickness cracks	$W \le 0.3\,mm \quad (0.0118\,in)$
For surface cracks	$W \le 0.5\,mm \quad (0.0197\,in)$

Mathematical prediction of the crack width is a very approximate process. Nevertheless, the method used for calculation of the crack width can have a significant impact on the resulting design. In some cases, the reinforcement requirement to control cracking is up to 30 percent higher than that required for satisfying the ultimate limit state. Therefore, considerable engineering judgment must be exercised to specify the allowable crack width limitations.

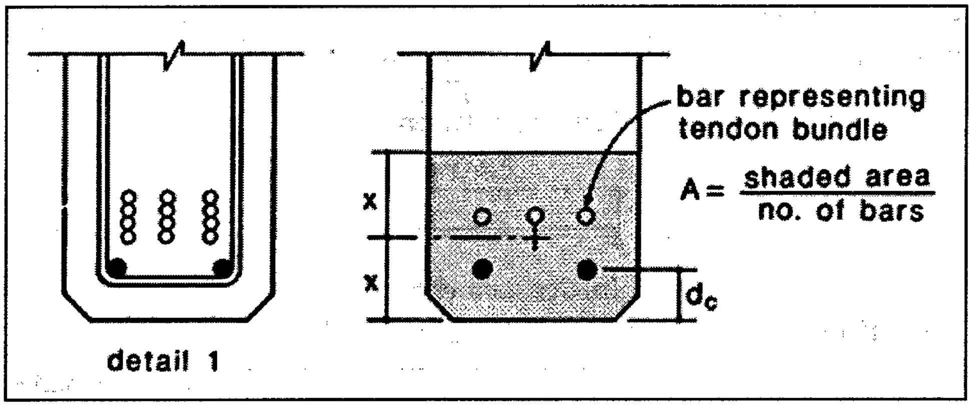


Figure 9.2. Definitions of d_c and A in crack width calculation

In summary, the crack control design should be based upon the experience and observation of the performance of existing structures in similar environments. For design of offshore structures, the general approach of using the allowable stress criteria is recommended for crack control. The crack width criteria are recommended only as a check in special areas, such as the splash zone, for critical structural members susceptible to fatigue damage, or for structural members that do not meet the allowable stress criteria.

9-4 Deflection

Both reinforced concrete components and prestressed concrete components shall be designed to have adequate stiffness to meet the short-term and long-term deflection requirements. Evaluation of deflection of P/C components shall take into account the effects of cracking, reinforcement, and time-dependent factors.

The moment causing flexural cracking due to external loading shall be calculated on the basis of the elastic properties, as follows:

$$M_{cr} = \frac{I_c}{y_t}(7.5\sqrt{f_c'} + f_{pe} - f_d) \tag{9-4}$$

For nonprestressed concrete, f_{pe} in Equation 9-4 is zero.

For nonprestressed members, short-term deflection shall be computed by usual elastic mechanics formula using the following effective moment of inertia (I_e) to account for effects of cracking and reinforcement:

$$I_e = \left[\frac{M_{cr}}{M_a}\right]^3 I_g + \left[1 - \left\{\frac{M_{cr}}{M_a}\right\}^3\right] I_{cr} \tag{9-5}$$

For nonprestressed members, long-term deflection in addition to the short-term deflection shall be computed by multiplying the short-term deflection by the factor λ:

$$\lambda = \frac{\xi}{1+50\rho'} \tag{9-6}$$

The value of ρ' should be the ratio of compression reinforcement at midspan for simple and continuous spans, and at support for cantilevers. The value of ξ may be taken from Figure 9.3.

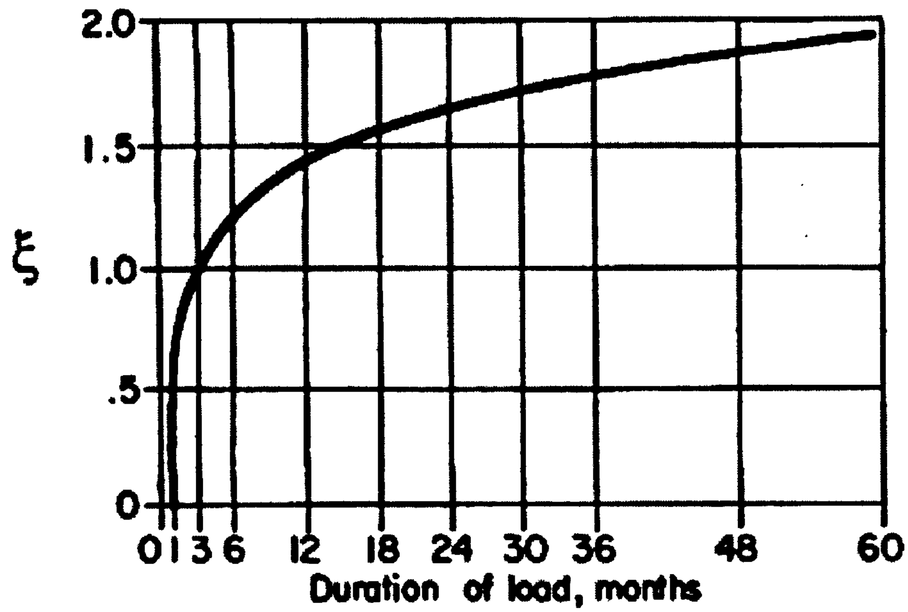


Figure 9.3. Values of time-dependent factor ξ

For prestressed concrete members, short-term deflection or camber shall be computed by usual elastic mechanics formula using the effective moment of inertia I_g. The long-term deflection or camber shall be computed by multiplying the short-term deflection by the factor C_2:

$$C_2 = \frac{C_1 + \frac{A_s}{A_p}}{1 + \frac{A_s}{A_p}} \tag{9-7}$$

Values of C_1 shall be taken from Table 9.6.

Table 9.6
Multiplier C_1 for Estimating Long-Term Deflection and Cambers in Prestressed Concrete Members

	Without Composite Topping	With Composite Topping
At erection:		
(1) Deflection (downward) component—apply to the elastic deflection due to the member weight at release of prestress	1.85	1.85
(2) Camber (upward) component—apply to the elastic camber due to prestress at the time of release of prestress	1.80	1.80
Final:		
(3) Deflection (downward) component—apply to the elastic deflection due to the member weight at release of prestress	2.70	2.20
(4) Camber (upward) component—apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.20
(5) Deflection (downward)—apply to elastic deflection due to superimposed dead load only	3.00	3.00
(6) Deflection (downward)—apply to elastic deflection caused by the composite topping	—	2.30

10 Strength Requirements

Notation

$\mathbf{A_{ps}}$ = area of prestressing steel in tension zone

$\mathbf{A_s}$ = area of mild steel tension reinforcement

$\mathbf{b}$ = cross-section width

$\mathbf{c}$ = distance from the extreme compression fiber to the neutral axis

$\mathbf{C}$ = concrete cover over reinforcement

$\mathbf{d_e}$ = corresponding effective depth from the extreme compression fiber to the centroid of the tensile reinforcement

$\mathbf{d_{eff}}$ = effective tension zone = $1.5C + 10d_b$

$\mathbf{d_b}$ = diameter of reinforcement

$\mathbf{d_p}$ = concrete cover to prestressing steel

$\mathbf{d_s}$ = concrete cover to mild reinforcing steel

$\mathbf{f'_c}$ = specified compressive strength of concrete

$\mathbf{f_{ps}}$ = average tensile stress in prestressing tendons on the tension side of the component

$\mathbf{f_t}$ = mean tensile strength of concrete

$\mathbf{f_y}$ = yield strength of the reinforcing steel

$\mathbf{f_{pe}}$ = effective prestress

$\mathbf{F_i}$ = impact load factor

$\mathbf{H_f}$ = hydraulic factor

$\mathbf{N_c}$ = the tension force due to unfactored dead plus live load

ρ_{min} = minimum reinforcement ratio

10-1 General

The load factors prescribed in EM 1110-2-2104 shall be applied to the ultimate strength design of precast/prestressed concrete components with the modifications described below.

The required strength against loads or load combinations during stages of fabrication, transportation, and erection of these members may be verified by the

load factor methods without use of the hydraulic factor H_f. However, an impact load factor $F_i = 1.15$ should be applied to proper loads to include the dynamic load amplification effect during transportation and erection of these members.

To check the strength of P/C members during underwater construction of hydraulic structures, the lateral pressure of underwater concrete against P/C members may be taken as the hydrostatic fluid pressure with a load factor of 1.4. Alternatively, time-dependent effects of the underwater concrete may be taken into account in calculation of the lateral pressure and a load factor of 1.7 shall be used.

For nonprestressed P/C members, the hydraulic factor $H_f = 1.3$ shall be applied only to the members that will be permanently exposed to abrasion/erosion action during the service life of the structure. The hydraulic factor should be applied for in-service loads only.

For prestressed concrete members, the hydraulic factor $H_f = 1.15$ shall be applied only to the members that will be permanently exposed to abrasion/erosion actions.

For nonprestressed precast members that are not exposed to abrasion/erosion action, the strength design need not include the hydraulic factor.

Table 10.1 illustrates an example of load factors for design of typical precast/prestressed concrete members under various load types and stages.

10-2 Strength Reduction Factors

The strength reduction factor shall comply with ACI 318, Provision 9.3.

10-3 Reinforcement Requirements

10-3.1 Material Properties

Mild steel reinforcement. The type and grade of reinforcing steel shall be limited to ASTM A 615 Grade 60 steel. Reinforcement of other grades and types permitted by ACI 318 may be permitted for special application subject to consultation and approval of Headquarters, USACE.

High-strength tendons and bars. Prestressing bars shall conform to ASTM A 722, with deformation conforming to ASTM A 615. Prestressing strands shall conform to ASTM A 416. Prestressing wires shall conform to ASTM A 421.

Table 10.1
Load Factors for Ultimate Strength Design of Precast Concrete

Load Description	Basic Load Factor	Hydraulic Factor	Minimum Dynamic Impact Factor	Notes[1]
Precast yard loads				
Dead load (on yard skids with dynamic factors)	1.4	1.0	1.15	
Post-tensioning jacking force for anchorage design	1.2	1.0	–	1
Form suction	1.4	1.0	–	
Transport loads				
Dead load (rigged in water with dynamic factor)	1.4	1.0	1.15	1
Hydrostatic pressure	1.4	1.0	–	
Dynamic inertial/drag (tow/current)	1.3	1.0	1.15	
Positioning loads				
Dead load (rigged in water on landing pads with dynamic factor)	1.4	1.0	1.15	1
Ballasting loads	1.7	1.0	–	
Hydrostatic pressure	1.4	1.0	–	
Dynamic inertial/drag (current)	1.3	1.0	1.15	
Construction loads				
Dead load	1.4	1.0	1.15	1
Hydrostatic pressure	1.4	1.0	–	
Hydrostatic pressure of underwater concrete	1.7	1.0	–	2
Thermal load due to hydration of cement	1.4	1.0	–	
Operation loading condition				
Permanent loads				
Dead loads	1.4	1.3	–	3,4
Earth pressure loads	1.4	1.3	–	3,4
Hydraulic loads				
Hydrostatic pressure	1.7	1.3	–	3,4
Dynamic inertial added mass	1.3	1.3	–	3,4
Flood stage	1.3	1.3	–	3
Base uplift pressure	1.7	1.3	–	3,4
Impact loads				
Barge impact load	1.3	1.3	1.0	3
Ice and debris impact load	1.3	1.3	1.0	3
Environmental loads				
Wind	1.3	1.3	–	3
Flow ice	1.3	1.3	1.0	3

[1] Notes:
1. Dead loads include any construction framing, and equipment attached to the members.
2. Use load factor 1.7 when pressure accounts for slump, temperature, pour rate, mix design, and consolidation procedures. Use load factor 1.4, when pressure is taken to be full equivalent hydrostatic head.
3. The hydraulic factor applies only to structural components that will be permanently exposed to abrasion/erosion action. For prestressed members, the hydraulic load factor of 1.15 may be adopted. For nonprestressed members subjected to direct tension, the hydraulic load factor is 1.65.
4. Basic load factors are reduced by 0.75, if combined with short-term dynamic loads.

10-3.2 Minimum Reinforcement for Prestressed Members

The minimum reinforcement ratio for prestressed concrete members shall be the larger of the following requirements.

For prestressed concrete members subjected to flexural bending, the maximum prestressing steel and reinforcement ratio shall be limited to the value such that

$$\frac{c}{d_e} \leq 0.42 \tag{10-1}$$

$$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y} \tag{10-2}$$

where f_{ps} should be calculated in accordance with Section 11.2.

For prestressed concrete members subjected to flexural bending, the minimum prestressing steel and reinforcement ratio shall be such that the flexural strength capacity at any section of a flexural component exceeds at least 1.2 times the cracking moment as defined in Section 9.4.

The minimum regular reinforcement ratio on the tension face of a prestressed concrete panel shall not be less than 0.0018 in each direction. If the panel is subjected to the through-cracking or stress reversals, the minimum reinforcement requirements shall be applied to both faces of a member.

10-3.3 Minimum Reinforcement for Precast Members

The minimum reinforcement ratio shall be the largest value of the following requirements:

For rectangular beam elements, $\rho_{\min} = \dfrac{200}{f_y}$.

For slab elements, $\rho_{\min} = 0.0028$, with one-half on each face in each way, but not exceeding a steel area equivalent to No. 9 bars at 12 in. (305 mm) in each face.

For wall elements, $\rho_{\min} = 0.001$, with one-half on each face in each way, but not exceeding a steel area equivalent to No. 9 bars at 12 in. (305 mm) in each face.

For slab in which tensile stress exceeds $2\sqrt{f_c'}$, reinforcing steel shall not be less than A_s, calculated as follows:

$$A_s = \frac{N_c}{0.5 f_y} \tag{10-3}$$

If tensile stress occurs on a face of a member during construction, transportation, and operation, the minimum steel area on the tensile face of the member should be calculated as follows:

$$A_s = \frac{f_t}{f_y} b d_{eff} \tag{10-4}$$

where d_{eff} should be taken as $1.5C + 10d_b$.

Maximum reinforcement ratio. The maximum reinforcement ratio shall comply with EM 1110-2-2104.

Spacing of reinforcement. Spacing of reinforcement at critical sections shall not exceed two times the member thickness. In general, reinforcement in P/C slabs and walls shall not be spaced farther apart than three times the member thickness, nor 18 in. (460 mm).

10-3.4 Anchorage for Tension Components

Tension components should be reliably anchored into the structure. A common way to make an anchorage is to use steel anchors, T-headed bars, or bolts. The anchorage capacity of the concrete should be checked based upon an assumed concrete cone model, as described in ACI 349, Appendix B ("Steel Embedments").

11 Flexural Design

Notation

A_{ps} = area of prestressing steel in tension zone (mm^2)
A_s = area of mild steel reinforcement in tension zone (mm^2)
A'_s = area of mild steel reinforcement in compression zone (mm^2)
a = depth of equivalent rectangular stress block
b = cross-section width of a rectangular section (mm)
b_w = cross-section width of the web of a T-section
c = distance from the extreme compression fiber to the neutral axis (mm)
d_p = concrete cover to prestressing steel (mm)
d_s = concrete cover to mild tension reinforcing steel (mm)
d_s' = concrete cover to mild compression reinforcing steel
f'_c = specified compressive strength of concrete (MPa)
f_{pe} = effective prestress (MPa)
f_{ps} = average tensile stress in prestressing tendons on the tension side of the component
f_{py} = specified yield strength of prestressing steel (MPa)
f_{pu} = specified tensile strength of prestressing steel (MPa)
f_y = yield strength of the tension reinforcing steel (MPa)
f_y' = yield strength of the compression reinforcing steel (MPa)
k = multiplier for calculation of prestressing stresses, depending only upon the type of prestressing tendons used
h_f = thickness of the flange in a T-section
M_n = nominal bending strength (MN-mm)
β_1 = adjustment factor for equivalent rectangular stress block, as defined in Equation 11-1

11-1 Design Basis

The flexural strength of both prestressed and nonprestressed concrete components shall be calculated on the basis of equilibrium and strain compatibility and the following assumptions:

Strain is directly proportional to the distance from the neutral axis, except for the deep beams.

The strain in a prestressing tendon is equal to the strain in the adjacent concrete fiber plus the difference in strain between the tendon and the adjacent concrete fiber.

If the concrete is unconfined, the maximum usable strain at the extreme concrete compression fiber is not greater than 0.003.

The tensile strength of concrete is neglected.

The concrete compressive stress-strain distribution is assumed to be a rectangular shape, with $0.85f_c'$ over the block bounded by the edges of the cross section and a straight line located parallel to the neutral axis at the distance $a = \beta_1 c$, from the extreme compression fiber.

$$\beta_1 = 0.85 - \frac{0.05(f_c'-28)}{7} \geq 0.65 \qquad (11\text{-}1)$$

11-2 Strength Analysis

This section addresses the flexural strength design for both prestressed and nonprestressed concrete members. In the following equations, f_{ps} and f_{pu} would be zero for nonprestressed concrete members.

For either a rectangular or flanged section subjected to flexure about one axis, the average stress in prestressing tendons shall be calculated as follows:

$$f_{ps} = f_{pu}(1 - k\frac{c}{d_p}) \qquad (11\text{-}2)$$

$$k = 2(1.04 - \frac{f_{py}}{f_{pu}}) \qquad (11\text{-}3)$$

Variable k may take the following values for various prestressing steel types conforming to ASTM standards without calculation:

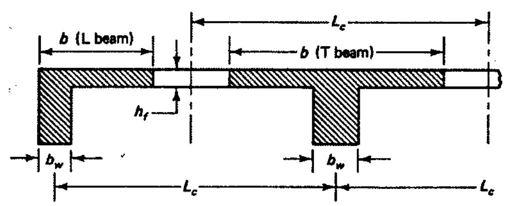
k = 0.28 for low-relaxation strands
= 0.38 for stress-relieved strands and plain high-strength bars
= 0.48 for deformed high-strength bars

For a flanged section with the compression flange depth less than c (i.e., $h_f > c$), the compression depth c and the nominal flexural strength shall be calculated as follows:

$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s' f_y' - 0.85 \beta_1 f_c'(b - b_w) h_f}{0.85 f_c' \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \tag{11-4}$$

$$M_n = A_{ps} f_{ps}(d_p - \frac{a}{2}) + A_s f_y (d_s - \frac{a}{2}) - A_s' f_y'(d_s' - \frac{a}{2}) + 0.85 f_c'(b - b_w) \beta_1 h_f (\frac{a}{2} - \frac{h_f}{2}) \tag{11-5}$$

For a rectangular section or flanged section with the compression flange depth greater than c (i.e., $h_f \leq c$, the compression depth c and the nominal flexural strength shall be calculated as follows:



$$c = \frac{A_{ps} f_{pu} + A_s f_y - A_s' f_y'}{0.85 f_c' \beta_1 b_w + k A_{ps} \dfrac{f_{pu}}{d_p}} \tag{11-6}$$

$$M_n = A_{ps} f_{ps}(d_p - \frac{a}{2}) + A_s f_y (d_s - \frac{a}{2}) - A_s' f_y'(d_s' - \frac{a}{2}) \tag{11-7}$$

Effective width, b, of flanged section shall be taken as the least of the following three values:

T- or I-section	L-section
$b_w + 6h_f$	$b_w + 12h_f$
½ $(b_w + L_c)$	L_c
$b_w + L/12$	$L/4$

12 Shear and Torsion

Notation

A_{cp} = total area enclosed by outside perimeter of concrete cross section in torsional strength calculation (mm^2)

A_o = area enclosed by the shear flow path and can be taken as $0.85A_{oh}$ (mm^2)

A_{oh} = area enclosed by center line of exterior closed transverse torsion reinforcement, including area of any holes (mm^2)

A_{ps} = area of prestressing steel in tension zone (mm^2)

A_s = area of mild steel reinforcement in tension zone (mm^2)

A'_s = area of mild steel reinforcement in compression zone (mm^2)

A_t = area of one leg of closed transverse torsion reinforcement (mm^2)

A_{total} = total shear web reinforcing steel area resisting both shear and torsion (N)

A_v = area of a transverse reinforcement within distance s (mm^2)

b_o = perimeter of the critical section surrounding the concentrated load area (mm)

b_v = effective section width (mm)

b_w = effective web width (mm)

c = distance from the extreme compression fiber to the neutral axis (mm)

d_v = effective shear depth taken as the distance between the resultants of the tensile and compressive forces due to flexure, but need not be less than $0.9d_e$ or $0.72h$

d_e = effective depth of the member (mm)

E_p = elastic modulus of prestressing steel

E_s = elastic modulus of mild reinforcing steel

f'_c = specified compressive strength of concrete (MPa)

f_{yv} = yield strength of the web reinforcing steel (MPa)

f_y = yield strength of the reinforcing steel (MPa)

f_{pc} = compressive stress in concrete after prestress losses have occurred either at the centroid of the cross section or at the junction of the web and flange where the centroid lies in the flange (MPa)

f_{pe} = effective prestress (MPa)

f_{py} = specified yield strength of prestressing steel (MPa)

f_{pu} = specified tensile strength of prestressing steel (MPa)

f_{p0} = stress in prestressing steel when the stress in the surrounding concrete is zero (MPa)

h = total depth of the member

M_n = nominal bending strength (N-mm)

M_u = factored moment (N-mm)

N_u = factored axial force, positive in tension (N)

p_c = length of the outside perimeter of the concrete section in torsional strength calculation (mm)

p_h = perimeter of the center line of the closed transverse torsion reinforcement (mm)

s = spacing of transverse reinforcement (mm)

s_x = a design parameter as defined in Figure 12.5

T_u = factor torsional force (N-mm)

T_{cr} = torsional cracking moment (N-mm)

v = calculated shear stress in the concrete (MPa)

V_u = factored shear load (N)

V_c = nominal shear strength of the concrete (N)

V_n = nominal shear strength of the reinforced concrete component (N)

V_s = nominal shear strength of mild shear reinforcement (N)

V_p = component of prestressing force in the direction of the shear force (N)

V_{total} = total shear force accounting for shear and torsion (N)

φ = strength reduction factor as defined in ACI 318, Provision 9.3

ε_x = strains in the longitudinal reinforcement

θ = angle of inclination of diagonal cracks to the longitudinal reinforcement (deg)

β = a factor indicating ability of tension transfer in diagonally cracked concrete

β_c = ratio of long side to short side of the rectangle through which the concentrated load or reaction force is transmitted

α = angle of inclination of transverse reinforcement to longitudinal axis (deg)

12-1 Design Basis

Shear strength shall be checked at any section of a member by comparing the factored shear force and the factored shear resistance. The nominal shear resistance of a section of a prestressed or reinforced concrete member shall be a sum of the shear resistance provided by concrete and the resistance by shear reinforcement and prestress:

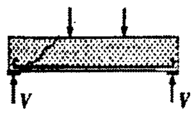
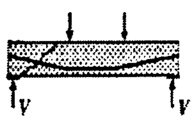
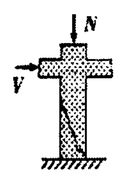
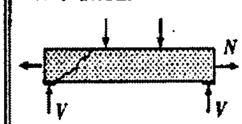
$$V_u \le \varphi V_n = \varphi\left(V_c + V_s + V_p\right) \tag{12-1}$$

The ACI shear design method and the AASHTO shear design method are both based upon the truss model. The truss model assumes that the shear resistance mechanism of a concrete member can be modeled by a truss analog in which bending moment and axial load are resisted by the chords of a truss, and shear is resisted by diagonal compression struts in the web.

In theory, the ACI shear design method is based upon calculation of the shear reinforcement contribution on the basis of the 45-deg truss equation. The AASHTO design method is based upon a variable truss model and the modified compression field theory. The modified compression field theory accounts for the influences of tensile variable inclination angle of shear cracks and the contribution of the longitudinal reinforcement to shear resistance of concrete.

From the viewpoint of design practices, the essential difference between the ACI shear design method and the AASHTO LRFD shear design method is complexity of the design procedures. The ACI 318 shear design provisions consist of more than 40 empirical equations for different structural members and different loading cases (some of which are shown in Figure 12.1), while the AASHTO LRFD shear design provisions basically entail four equations that cover all the loading cases for both prestressed and nonprestressed concrete members. Figure 12.1 shows a comparison between the ACI shear design method and the AASHTO shear design method (so-called "General Method" in right column).

Both the ACI design method and AASHTO design method produce acceptable results for P/C thin-wall structures. The AASHTO design method is presented here because of the simplicity of its design procedure.

	ACI Method	General Method
	$V_n = V_c + V_s$	$V_n = V_c + V_s + V_p$
Non-Prestressed Beams 	$V_c = \left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u}\right) b_w d$ but $\frac{V_u d}{M_u} \le 1.0$ $V_c \le 3.5\sqrt{f'_c}\, b_w d$ or $V_c = 2\sqrt{f'_c}\, b_w d$ $V_s = \frac{A_v f_y d}{s}$ $\quad V_s \le 8\sqrt{f'_c}\, b_w d$	$V_c = \beta\sqrt{f'_c}\, b_v d_v$ $V_s = \frac{A_v f_y}{s} d_v \cot\theta$ where β and θ are functions of the strain, ϵ_x, shear stress, v, and crack spacing s_x where $v = \frac{V_u - V_p}{b_v d_v}$ and $\epsilon_x = \frac{M_u/d_v + 0.5(N_u + V_u\cot\theta) - A_{ps} f_{po}}{E_s A_s + E_p A_p}$
Prestressed Beams 	$V_c = \left(0.6\sqrt{f'_c} + 700\frac{V_u d}{M_u}\right) b_w d$ but $2\sqrt{f'_c}\, b_w d \le V_c \le 5\sqrt{f'_c}\, b_w d$ or $V_c = V_{ci} = 0.6\sqrt{f'_c}\, b_w d + V_d + \frac{V_i M_{cr}}{M_{max}}$ but $V_{ci} \ge 1.7\sqrt{f'_c}\, b_w d$ and $V_c \le V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right) b_w d + V_p$ $V_s = \frac{A_v f_y d}{s} \le 8\sqrt{f'_c}\, b_w d$	
Axial Compression and Shear 	$V_c = \left(1.9\sqrt{f'_c} + 2500\rho_w \frac{V_u d}{M_u - N_u\frac{(4h - d)}{8}}\right) b_w d$ $V_c \le 3.5\sqrt{f'_c}\, b_w d\sqrt{1 + \frac{N_u}{500 A_g}}$ $V_s = \frac{A_v f_y d}{s} \le 8\sqrt{f'_c}\, b_w d$	
Axial Tension and Shear 	$V_c = 2\left(1 + \frac{N_u}{500 A_g}\right)\sqrt{f'_c}\, b_w d$ $V_s = \frac{A_v f_y d}{s} \le 8\sqrt{f'_c}\, b_w d$	

Detailing Rules (ACI Method)

- Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of the member or $12d_b$, which is greater,..
- Flexural reinforcement shall not be terminated in a tension zone unless
  - shear at cutoff ≤ 2/3 shear permitted, or
  - stirrup area, A_v, in excess of that required for shear and torsion, is provided ... ... $A_v \ge 60 b_w s/f_y$ ... $s \le d/8\beta_b$, or
  - for #11 bars or smaller: shear at the cutoff ≤ 3/4 shear permitted and continuing reinforcement provides double the area required for flexure at the cutoff.
- At simple supports and points of inflection, the diameter of the positive moment tension reinforcement shall be limited so that

$$l_d \le \frac{M_n}{V_u} + l_a$$

Detailing Rules (General Method)

Longitudinal steel must be detailed so that

$$A_s f_y + A_{ps} f_{ps} \ge \frac{M_u}{\phi d_v} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

Figure 12.1. Comparison of the shear design methods (ACI 318 versus AASHTO LRFD)

12-2 Shear Design for One-Way Action

For one-way shear action, the nominal shear resistance provided by concrete shall be calculated as follows:

$$V_c = 0.083\beta\sqrt{f_c'}b_v d_v \tag{12-2}$$

The shear resistance provided by concrete and shear reinforcement shall not exceed $0.25f_c' b_v d_v$, as in

$$V_c + V_s \leq 0.25 f_c' b_v d_v \tag{12-3}$$

The shear reinforcement shall be calculated as follows:

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \tag{12-4}$$

where α is the angle of inclined shear reinforcement to the horizontal axis. For vertical shear reinforcement, the shear reinforcement shall be calculated as follows:

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \tag{12-5}$$

where θ is the angle of inclination of diagonal cracks to the longitudinal reinforcement, and β is a factor indicating ability of tension transfer in diagonally cracked concrete (see Figure 12.2). Both β and θ should be determined on the basis of the strains in the longitudinal reinforcement (ε_x) and the shear stress in concrete (v) in accordance with Figures 12.3 and 12.4 for members with transverse shear reinforcement and without transverse shear reinforcement, respectively. Alternatively, β and θ may be determined from Tables 12.1 and 12.2 for members with transverse shear reinforcement and without transverse shear reinforcement, respectively. Figure 12.5 illustrates the physical parameter s_x in Table 12.2.

The strain in the longitudinal steel at the cracks shall be calculated as follows:

$$\varepsilon_x = \frac{\dfrac{M_u}{d_v} + 0.5N_u + 0.5V_u \cot\theta - A_{ps} f_{p0}}{E_s A_s + E_p A_{ps}} \leq 0.002 \tag{12-6}$$

where f_{p0} is stress in prestressing steel when the stress in the surrounding concrete is zero. Alternatively, f_{p0} can be conservatively taken as the effective prestress after losses, f_{pe}.

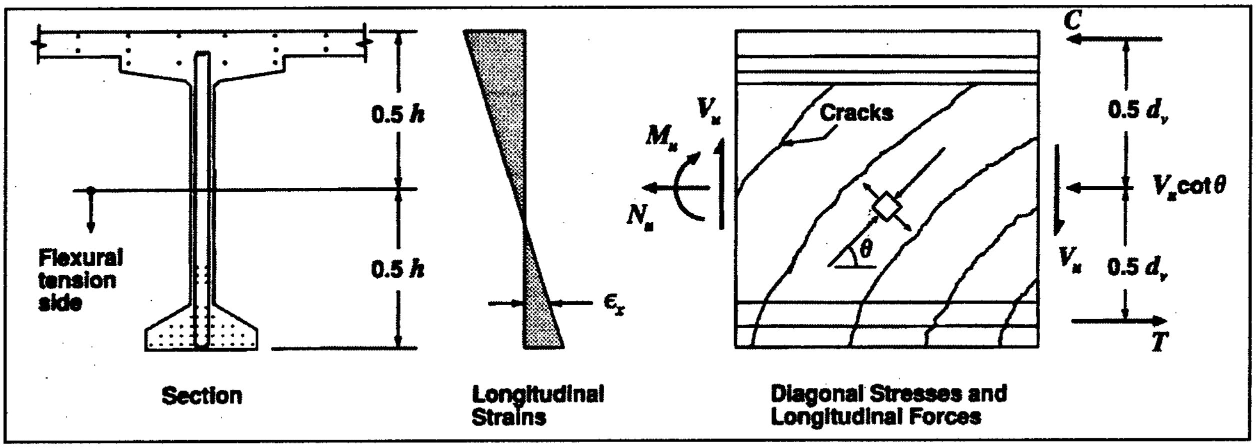


Figure 12.2. Illustration of shear design parameters

The shear stress in concrete should be calculated as follows:

$$v = \frac{V_u - \varphi V_p}{\varphi b_v d_v} \qquad (12\text{-}7)$$

The design parameters β and θ should be obtained from the calculated ε_x and v and from the table and the figure below.

When the member is not subjected to torsion, the minimum longitudinal reinforcement should be as follows:

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v \varphi} + 0.5\frac{N_u}{\varphi} + \left(\frac{V_u}{\varphi} - 0.5V_s - V_p\right)\cot\theta\right] \qquad (12\text{-}8)$$

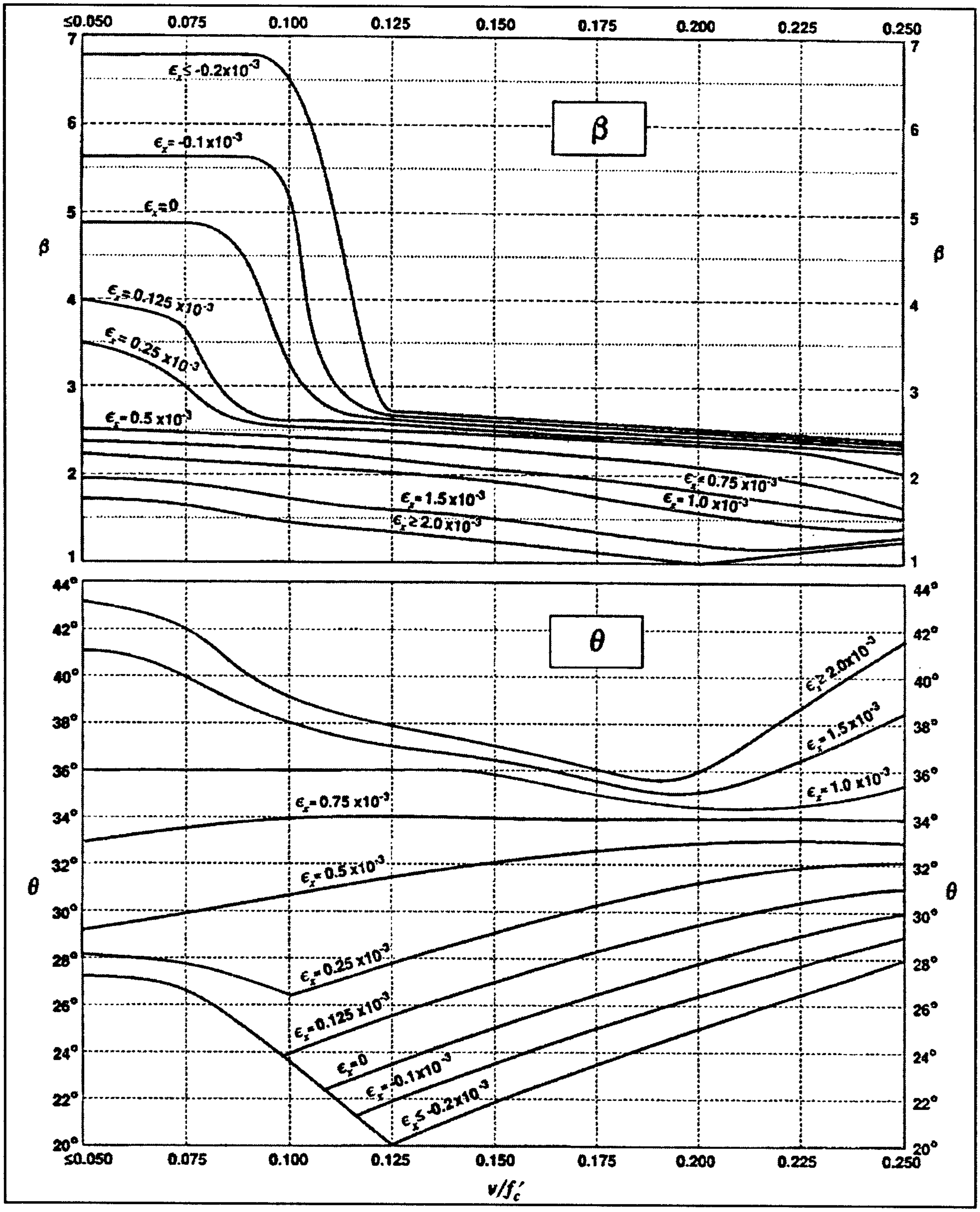


Figure 12.3. Values of θ and β for sections with transverse reinforcement

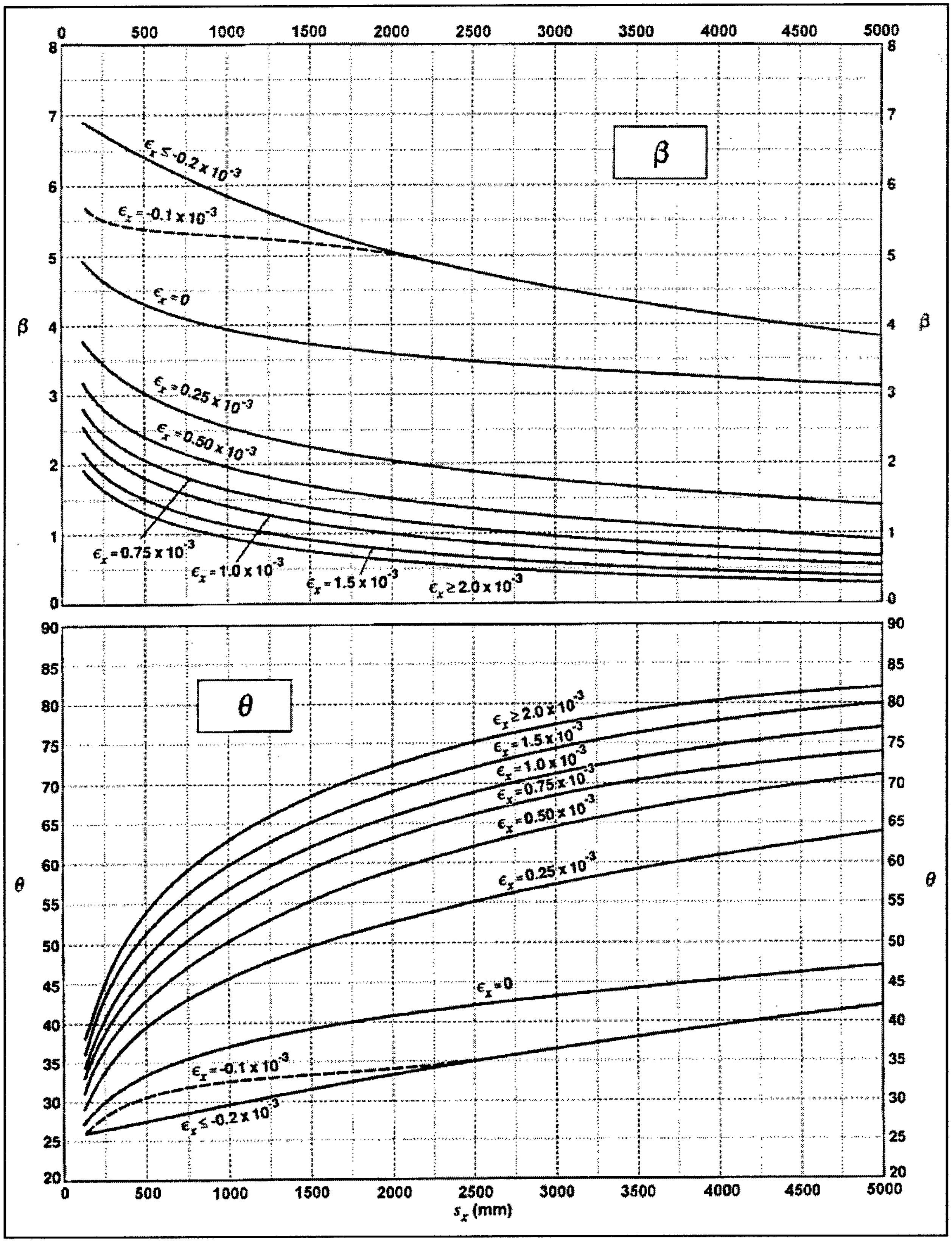


Figure 12.4. Values of θ and β for sections without transverse reinforcement

$\frac{v}{f'_c}$	$\varepsilon_x \times 1000$										
	-0.2	-0.15	-0.1	0	0.125	0.25	0.5	0.75	1	1.5	2
<= 0.05	27.0 6.78	27.0 6.17	27.0 5.63	27.0 4.88	27.0 3.99	28.5 3.49	29.0 2.51	33.0 2.37	36.0 2.23	41.0 1.95	43.0 1.72
0.075	27.0 6.78	27.0 6.17	27.0 5.63	27.0 4.88	27.0 3.65	27.5 3.01	30.0 2.47	33.5 2.33	36.0 2.16	40.0 1.90	42.0 1.65
0.1	23.5 6.50	23.5 5.87	23.5 5.31	23.5 3.26	24.0 2.61	26.5 2.54	30.5 2.41	34.0 2.28	36.0 2.09	38.0 1.72	39.0 1.45
0.125	20.0 2.71	21.0 2.71	22.0 2.71	23.5 2.60	26.0 2.57	28.0 2.50	31.5 2.37	34.0 2.18	36.0 2.01	37.0 1.60	38.0 1.35
0.15	22.0 2.66	22.5 2.61	23.5 2.61	25.0 2.55	27.0 2.50	29.0 2.45	32.0 2.28	34.0 2.06	36.0 1.93	36.5 1.50	37.0 1.24
0.175	23.5 2.59	24.0 2.58	25.0 2.54	26.5 2.50	28.0 2.41	30.0 2.39	32.5 2.20	34.0 1.95	35.0 1.74	35.5 1.35	36.0 1.11
0.2	25.0 2.55	25.5 2.49	26.5 2.48	27.5 2.45	29.0 2.37	31.0 2.33	33.0 2.10	34.0 1.82	34.5 1.58	35.0 1.21	36.0 1.00
0.225	26.5 2.45	27.0 2.44	27.5 2.43	29.0 2.37	30.5 2.33	32.0 2.27	33.0 1.92	34.0 1.67	34.5 1.43	36.5 1.18	39.0 1.14
0.25	28.0 2.36	28.5 2.36	29.0 2.32	30.0 2.30	31.0 2.28	32.0 2.01	33.0 1.64	34.0 1.52	35.5 1.40	38.5 1.30	41.5 1.25

Table 12.1. Values of θ and β for Sections with Transverse Reinforcement

s_x	$\varepsilon_x \times 1000$								
	-0.2	-0.1	0	0.25	0.5	0.75	1	1.5	2
<= 130	26.0 6.90	26.0 5.70	27.0 4.94	29.0 3.78	31.0 3.19	33.0 2.82	34.0 2.56	36.0 2.19	38.0 1.93
250	27.0 6.77	28.0 5.53	30.0 4.65	34.0 3.45	37.0 2.83	39.0 2.46	40.0 2.19	43.0 1.87	45.0 1.65
380	27.0 6.57	30.0 5.42	32.0 4.47	37.0 3.21	40.0 2.59	43.0 2.23	45.0 1.98	48.0 1.65	50.0 1.45
630	28.0 6.24	31.0 5.36	35.0 4.19	41.0 2.85	45.0 2.26	48.0 1.92	51.0 1.69	54.0 1.40	57.0 1.18
1270	31.0 5.62	33.0 5.24	38.0 3.83	48.0 2.39	53.0 1.82	57.0 1.50	59.0 1.27	63.0 1.00	66.0 0.83
2500	35.0 4.78	35.0 4.78	42.0 3.47	55.0 1.88	62.0 1.35	66.0 1.06	69.0 0.87	72.0 0.65	75.0 0.52
5000	42.0 3.83	42.0 3.83	47.0 3.11	64.0 1.39	71.0 0.90	74.0 0.66	77.0 0.53	80.0 0.37	82.0 0.28

Table 12.2. Values of θ and β for Sections without Transverse Reinforcement

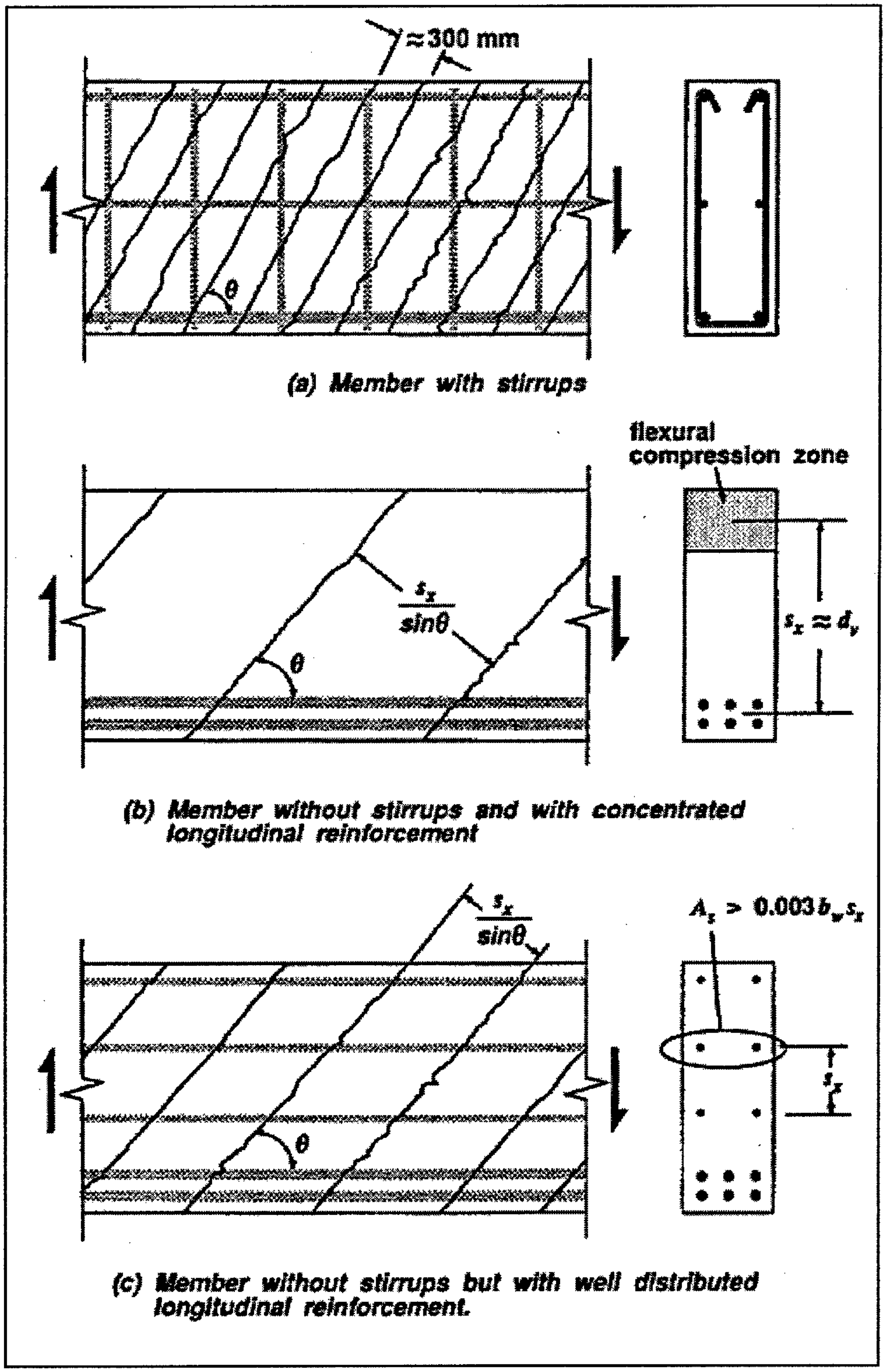


Figure 12.5. Illustration of s_x in shear design

12-3 Shear Design for Two-Way Actions

When thin plates are subject to concentrated loads or reaction forces, the shear strength should be checked for the more critical condition of the following two situations:

(1) *One-way shear action*: the critical section should be taken at distance d from the face of the concentrated load or reaction area. (Variable d is taken as the overall depth of the plate.) Shear strength design for one-way action should follow the procedure in Section 12-2.

(2) *Two-way shear action*: the critical section should be located so that its perimeter, b_o, is a minimum but not closer than $0.5d$ to the perimeter of the concentrated load, or reaction area, and from the face of any change in the slab thickness. Shear strength design for two-way action should follow the procedure described in this section.

For sections without transverse reinforcement, the nominal shear strength for two-way action V_n (in N) should be calculated as follows:

$$V_n = \left(0.17 + \frac{0.33}{\beta_c}\right)\sqrt{f_c}\,b_o d_v \leq 0.33\sqrt{f'_c}\,b_o d_v \tag{12-9}$$

when $V_u > \varphi V_n$, shear reinforcement, should be added to increase the shear strength at the critical section, using the calculation procedure in Section 12-2 with angle θ taken as 45 deg.

For sections with transverse reinforcement, the nominal shear strength under two-way shear action should be calculated as follows:

$$V_n = V_c + V_s = 0.166\sqrt{f_c'}\,b_o d_v + \frac{A_v f_y d_v}{s} \tag{12-10}$$

The maximum nominal shear strength shall be limited as follows:

$$V_n \leq 0.504\sqrt{f_c'}\,b_o d_v \tag{12-11}$$

12-4 Torsion Design

Both the torsional strength of concrete and the web stirrups resist the factored torsional moment in a member:

$$T_u \leq \varphi\left(T_c + T_s\right) \tag{12-12}$$

The torsional moment that causes cracking of concrete shall be determined as follows:

$$T_{cr} = 4\sqrt{f_c'}\frac{A_{cp}^{2}}{p_c}\sqrt{1+\frac{f_{cp}}{4\sqrt{f_c'}}} \quad (12\text{-}13)$$

where f_{cp} is compressive stress in concrete after prestress losses. For nonprestressed members, $f_{cp} = 0$.

Adequate web reinforcement shall be provided to resist the torsional moment, if the torsional moment is greater than 25 percent of the cracking torsional moment capacity of the members, as follows:

$$T_u \geq 0.25\varphi T_{cr} \quad (12\text{-}14)$$

Then, the web reinforcing steel area as required to resist the torsion shall be calculated with the following equation:

$$T_u \leq \varphi T_n = \varphi\frac{2A_o A_t f_y \cot\theta}{s} \quad (12\text{-}15)$$

where A_o is the area enclosed by the shear flow path and can be approximately taken as $0.85A_{oh}$.

The inclination angle θ must be determined on the basis of the strains in the longitudinal reinforcement (ε_x) and the shear stress in concrete due to both shear and torsion.

12-5 Shear Plus Torsion Design

Total shear web steel area is a sum of the steel area required for shear and the steel area required for torsion:

$$A_{total} = A_v + A_t \quad (12\text{-}16)$$

where A_v and A_t are web reinforcement areas required for resisting shear and torsion, and shall be determined in accordance with Section 12.2 and Section 12.3, respectively, with the following exceptions:

(1) For determination of ε_x, the total shear force due to direct shear and torsion shall be used to replace the factored shear as follows:

$$V_{total} = \sqrt{V_u^{\,2} + \left[\frac{0.9\,p_h T_u}{2A_o}\right]^2} \quad (12\text{-}17)$$

(2) For determination of the inclination θ, the shear stress in concrete (v) due to both shear and torsion shall be calculated as follows:

a. For box sections, the shear stress due to shear force and the shear stress due to torsion should be directly added together to compute the maximum shear stress on one side of the box member as follows:

$$v = \frac{V_{total} - \varphi V_p}{\varphi b_v d_v} + \frac{T_u p_h}{\varphi A_{oh}^2} \tag{12-18}$$

where p_h is perimeter of the center line of the closed transverse torsion reinforcement.

b. For other sections, the maximum shear stress in concrete should be calculated by combining the shear stress due to shear force and the shear stress due to torsion in accordance with the "root-mean-square" rule as follows:

$$v = \sqrt{\left[\frac{V_{total} - \phi V_p}{\phi b_v d_v}\right]^2 + \left[\frac{T_u p_h}{\phi A_{oh}^2}\right]^2} \tag{12-19}$$

When the member is subjected to a combination of shear, torsion, and flexure, the minimum longitudinal reinforcement should be as follows:

$$\phi\left(A_s f_y + A_{ps} f_{ps}\right) \geq \frac{M_u}{d_v} + 0.5N_u + \cot\theta\sqrt{\left(V_u - 0.5V_s - V_p\right)^2 + \left(\frac{0.45 p_h T_u}{2A_o}\right)^2} \tag{12-20}$$

Minimum transverse closed stirrups:

$$A_v + 2A_t = \frac{50 b_w s}{f_{yv}} \tag{12-21}$$

13 Composite Construction

Notation

$\mathbf{A_c}$ = area of composite contact surface being investigated for horizontal shear (mm^2)

$\mathbf{A_{pc}}$ = cross-sectional area of the precast concrete (mm^2)

$\mathbf{A_v}$ = steel area of cross ties on the composite interface (mm^2)

$\mathbf{b}$ = width of the infill concrete (mm)

$\mathbf{b_v}$ = width of precast concrete segment at the interface

$\mathbf{b_m}$ = modified width of the section (mm)

$\mathbf{d}$ = effective depth from the extreme compression fibers of the composite section to the centroid of the steel reinforcement (mm)

$\mathbf{e}$ = eccentricity of prestress tendons at the section (mm)

$\mathbf{E_{ci}}$ = elastic modulus of the infill concrete (MPa)

$\mathbf{E_{cp}}$ = elastic modulus of precast concrete (MPa)

$\mathbf{f}$ = stresses at the extreme fibers of the precast concrete section (MPa)

$\mathbf{f_y}$ = yield strength of the reinforcing steel (MPa)

$\mathbf{l_{vh}}$ = length of shear transfer between two sections of composite member (mm)

$\mathbf{M_d}$ = moment due to self-weight of the precast concrete

$\mathbf{M_i}$ = moment caused by the dead load and thermal load of the infill concrete and all the construction loads at the time of placement

$\mathbf{M_s}$ = moment caused by the critical service loads

$\mathbf{N}$ = ratio of the elastic modulus of infill concrete and the elastic modulus of precast concrete

$\mathbf{P_e}$ = effective prestress at the construction stage (MN)

$\mathbf{s}$ = spacing of the cross ties (mm)

$\mathbf{S_{pc}}$ = section modulus of the precast concrete (mm^3)

$\mathbf{S_c}$ = section modulus of the composite transformed sectional modulus with respect to the extreme fiber under consideration (mm^3)

$\mathbf{V_u}$ = factored shear at section considered (N)

$\mathbf{V_{nh}}$ = nominal shear strength at the interface of the composite segments (N)

μ = coefficient of friction at the interface

λ = adjustment factor for friction accounting for the concrete type

φ = strength reduction factor as defined in Chapter 10

ρ_v = ratio of tie reinforcement area to area of contact surface = $A_v/b_v s$

13-1 General

Provisions of this section for composite structural members shall apply to precast concrete and cast-in-place concrete elements that are constructed in separate placements but are designed to respond to loads as a unit. Therefore, the entire composite member shall be designed for resisting shears and moments that will be encountered during the service conditions.

In addition, P/C elements shall be designed for all the loads that will be encountered during construction, including transportation and installation loads, thermal stresses, and hydrostatic form pressure from infill concrete.

13-2 Design Basis

Composite members shall be designed for serviceability and ultimate strengths. The ultimate flexural and shear strength of composite members should account for different strengths of the P/C and the infill concrete. In calculating the ultimate strength of the final composite section, the strain discontinuity at the interface of both concretes and the loading sequences can be ignored to simplify the design. Studies show that this simplification produces satisfying design that is generally consistent with relevant test results.

Serviceability requirements include crack width calculation for nonprestressed composite members and concrete stress check for prestressed concrete composite members. The crack width check shall follow the design procedure outlined in Section 9.3. The crack width may be calculated for the service conditions only. However, the residual stresses that are locked in the P/C during placement of the infill concrete shall be considered.

For prestressed composite concrete members, the need to control the tensile stresses on the external concrete surface often governs the choice of the prestressing force. The Working Stress Method shall check the stresses on the external surfaces of composite members against the permissible stress criteria. The stress calculation must take account the strain compatibility between both concretes and all critical loads during the construction process, including hydrostatic form pressure and thermal stresses of the infill concrete.

13-3 Composite Transformed Cross Sections

If the infill concrete is of different strength than the precast concrete, a composite transformed cross section shall be used to account for the difference in the elastic moduli of the two concretes to ensure that the strains in both materials

at the interface are compatible. The composite transformed section is obtained by modifying the width of the infill concrete as follows:

$$b_m = \frac{E_{ci}}{E_{cp}} b = nb \tag{13-1}$$

The modified width b_m shall be used to calculate composite transformed section properties.

13-4 Flexural Strength

Flexural strength of the composite members shall be designed in accordance with the design procedure for T-beams or I-beams (outlined in Section 10) except that the infill concrete section is assumed to have the modified width, as defined in Section 13.3. The entire composite section of the member is assumed to have the strength of the precast concrete. The width of the web in the design equations for T-beams in Section 13.4 shall be replaced with b_m above.

In addition, the stresses in the extreme fibers of the concrete members should be checked against the permissible stress criteria for both the construction loads and the service loads as follows:

Concrete stresses at the construction stages prior to placing infill concrete:

$$f = -\frac{P_e}{A_{pc}} \pm \frac{P_e e}{S_{pc}} \mp \frac{M_d + M_i}{S_{pc}} \tag{13-2}$$

Concrete stresses under the service load condition:

$$f = -\frac{P_e}{A_{pc}} \pm \frac{P_e e}{S_{pc}} \mp \frac{M_d + M_i}{S_{pc}} \mp \frac{M_s}{S_c} \tag{13-3}$$

13-5 Shear Strength

Shear strength of composite members shall be calculated in accordance with the provisions in Chapter 12. The entire cross section of the composite members shall be used in the calculation. If the infill concrete is of different strength than the precast concrete, the lower concrete strength shall be assumed in the design.

If shear reinforcement is required to resist critical combinations of shear and torsion, the reinforcement shall extend across the entire composite section and be fully anchored in the precast concrete.

13-6 Shear Transfer Over the Interface

Shear strength on the interface of the P/C and the infill concrete shall be checked to ensure full shear transfer between segments of composite members. The shear strength calculation shall be based on either the direct design method or the basic design method described below.

(1) *Direct design method*:

$$V_u \le \varphi V_{nh} \tag{13-4}$$

The value of V_{nh} should be determined as follows:

a. When no cross ties are provided at the interface but the surface of the P/C segment is intentionally roughened or corrugated:

$$V_{nh} = 0.6 b_v d \tag{13-5}$$

b. When minimum cross ties are provided, but the contact surface of the precast segment is not intentionally roughened or corrugated:

$$V_{nh} = 0.6 b_v d \tag{13-6}$$

The minimum amount of cross ties shall have the following steel area:

$$A_v = \frac{b_v s}{3 f_y} \tag{13-7}$$

Spacing of ties shall not exceed four times the least dimension of the precast segment, nor 600 mm (24 in.).

c. When the contact surface of the precast segment is roughened or corrugated to a full amplitude of 5 mm (0.25 in.) and the minimum cross ties are provided:

$$V_{nh} = \left(1.8 + 0.6 \rho_v f_y\right) \lambda b_v d \le 3.5 b_v d \tag{13-8}$$

where λ is 1.0 for normal-weight concrete, 0.85 for sand-lightweight concrete, and 0.75 for all-lightweight concrete.

d. If the factored shear $V_u \ge 3.5 b_v d$, the shear friction method shall be used to design the shear reinforcement as follows:

$$V_{nh} = \mu A_v f_y \tag{13-9}$$

where μ is coefficient of friction at the interface and is determined from the following table:

Concrete Interface Conditions	μ
Concrete placed against hardened concrete surface roughened to an amplitude of 0.25 in. (6 mm)	1.0λ
Concrete placed against hardened concrete surface that is not intentionally roughened	0.6λ

(2) *Basic design method*:

As an alternative method, the shear strength can be checked by considering the shear actually transferred to the interface. The shear force shall be determined by computing the actual change in compressive or tensile force in precast segment between any two sections. That net change in the force shall be adequately transferred across the interface to the support segment. In general, the shear force transferred between the maximum moment section and the adjacent zero-moment section is often assumed to be the critical force. If shear friction is used, the total area of steel within the plane can be used to resist the shear force.

Nominal shear strength shall be calculated in accordance with the provisions for the direct design method except that the actual contact area of the interface shall be used in lieu of effective section area ($b_v d$). The contact area can be defined as

$$A_c = b_v l_{vh} \tag{13-10}$$

where l_{vh} is the length of shear transfer between two sections of composite member, usually between the maximum moment section and the adjacent zero-moment section.

13-7 Deflection Analysis

The deflection of composite members shall be estimated using the multiplier method defined in Section 9-4, except that the moment of inertia shall be calculated on the basis of the composite transformed section.

14 Fatigue Strength Design

Notation

$\mathbf{C_1}$ = environmental factor

$\mathbf{f_c'}$ = specified compressive strength of concrete

$\mathbf{f_{rd}}$ = design strength corresponding to the referenced failure mode

$\mathbf{j}$ = total number of load blocks considered

$\mathbf{n_i}$ = actual number of load cycles for load block I

$\mathbf{N_i}$ = number of load cycles causing failure if load block i acts alone

$\boldsymbol{\sigma_{max}}$ = largest compressive stress calculated as the mean stress within each stress block

$\boldsymbol{\sigma_{min}}$ = smallest compressive stress calculated as the mean stress within each stress block (for tensile stress, $\sigma_{min} = 0$)

$\boldsymbol{\eta}$ = Minor's sum

14-1 General

When critical P/C components are subjected to cyclic loads or repeated impact loads such as lock guard wall panels, fatigue resistance is likely to be a serious problem. These concrete structures shall be designed against fatigue failure. The objective of the fatigue strength design is to establish an acceptable safety margin against cyclic loading of variable magnitudes within the service life of the structures. In the fatigue analysis, the load factor is taken as 1.0. The modulus ratio of steel to concrete, E_s/E_c, is commonly taken as 10 to reflect the degradation of concrete stiffness under cyclic loading. All analyses should be based upon transformed section properties according to elastic theory.

In design of concrete structures, two design methods are available for the evaluation of structural components against fatigue failure: stress limit control method and comprehensive fatigue analysis. The stress limit control method is recommended for routine fatigue strength check of concrete components. In general, the comprehensive fatigue analysis approach is substantially more complex than the stress limitation control method. Thus, applications of comprehensive fatigue analysis in practical design are limited to special structures or critical structural components for which fatigue is likely to be a serious problem.

Fatigue analysis shall be conducted with caution and sound engineering judgment, because the analysis method (either the stress limit control or the comprehensive fatigue analysis) is empirical in nature. The method provides merely lower bounds of widely scattered empirical data. In addition, the analysis does not reflect the effects of loading sequence, random variations in stress ranges, and rest periods.

Fatigue design shall impose special design detailings against potential fatigue failure, including the following requirements:

a. No more than 30 percent of the longitudinal bars shall be spliced at a single location. Concentration of splices will cause high stress concentration, leading to cracking, loss of the bond, and progressive bond slip.

b. If lap splices of reinforcement or pretensioning anchorage are subjected to cyclic tensile stresses greater than 50 percent of the allowable static stresses, the lap length or prestressing development length shall be increased by 50 percent.

c. If splice of reinforcement will be subjected to repeated cyclic tension loads, through-member confinement reinforcement shall be provided at the splices.

14-2 Fatigue Failure Mechanisms of Concrete Structures

Fatigue failure of concrete is caused primarily by progressive internal microcracking. The microcracks usually initiate at the aggregate-to-paste interface and spread around the aggregates into the concrete matrix. Intensive development of internal cracking prior to failure causes a significant increase in both longitudinal and transverse deformation. When compressive stress reaches $0.7f'_c$, microcracking initiates and stiffness decreases, leading to potential dynamic amplification under cyclic loading. There is evidence that Poisson's ratio also increases under the circumstances. Unlike compression failure of concrete under static loading, fatigue failure of concrete in compression is ductile in nature. The local concentration of compressive stress under repeated loading will be relieved and redistributed prior to rupture.

Tensile cracking is initiated when the tensile strength of the concrete is exceeded, either by excessive stress excursions into the tensile range which overcome both the prestress and the static strength of the concrete, or by repeated cycling leading to tensile fatigue of the concrete. Tests show that cyclic loading at about 50 percent of the static tensile strength of the concrete can cause fatigue cracking.

Although concrete does suffer progressive loss of strength with increasing numbers of loading cycles, a comparison of the *S-N* curves with probable distributions of structural stresses during service life of hydraulic concrete structures shows extremely low probability of cumulative damage at the high-cycle end of the load spectrum. For a typical hydraulic concrete structure, however, significant damage can occur at the low-cycle, high-amplitude end of the load spectrum. That is, a relatively small number of load cycles of high magnitude can cause a sizable reduction in stiffness and rapid increases in strains that lead to cracking and spalling.

There is a significant difference in fatigue endurance between uncracked concrete and cracked concrete that is subjected to continuous crack opening and closing. When concrete is subjected to excursion into the tensile range of the concrete, but still without cracking, the allowable maximum stress range can be determined by a modified Goodman diagram as shown in Figure 14.1.

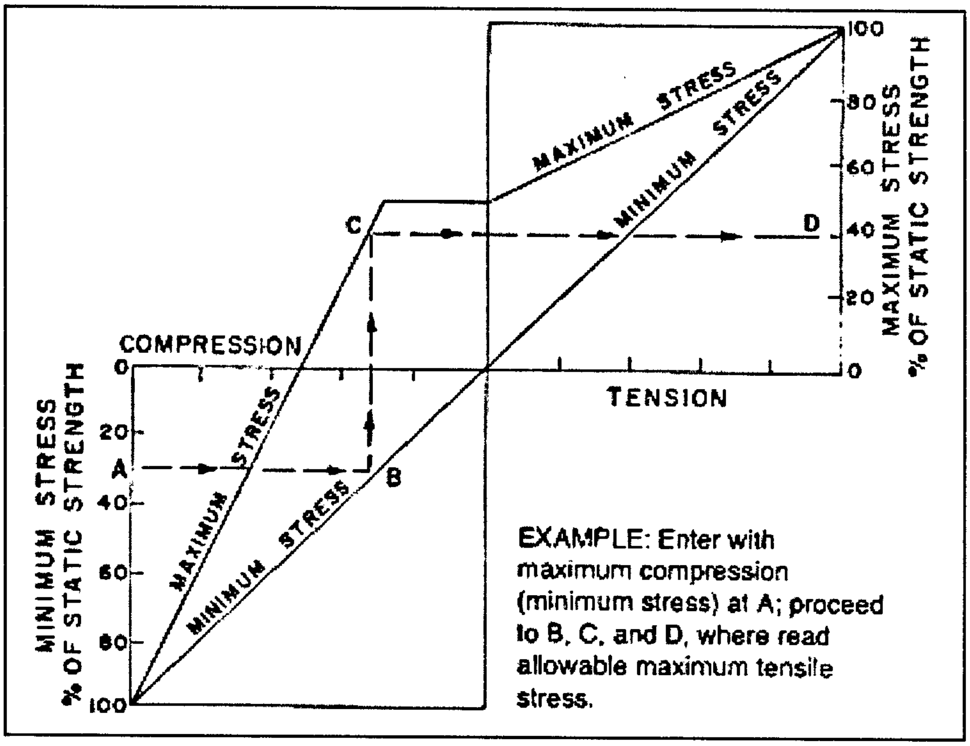


Figure 14.1. Modified Goodman diagram for submerged concrete subjected to fully reversing stresses, 90-percent probability of nonfailure under 3×10^6 cycles

In hydraulic concrete structures, cracking may occur due to overload, accident, construction procedure, creep, and thermal strains. Without effective prestress, the cracks can be repeatedly opened and closed by the subsequent cyclic loads. Alternating opening and closing of cracks causes several adverse structural responses. First of all, the reinforcing steel across cracks must pick up the tension force previously carried by the tensile capacity of concrete. Initially, "tension stiffening" of the concrete may somewhat alleviate the stress jump in steel at cracks. But the tension stiffening is degraded rapidly under cyclic loading. As the cracks alternately open and close, there is also a significant jump

in the concrete stress. This stress increase in concrete is due to the dynamic hammering effect on crack closing and also to stress amplification under decreasing stiffness. The dynamic effect of the crack closing ("hammering") leads to mechanical abrasion and breaking loose of the aggregate particles. The net result is that, at cracking, there is a drastic jump in the stress range of the steel, combined with a significant increase in the maximum compression stress level in the concrete.

The repeated excursion into the tensile cracking is especially detrimental for submerged concrete. Opening and closing of the crack cause the pumping of water in and out of the crack. Water in the cracks is subjected to high temporary pressures during crack closing. As the water exits the crack at high velocity, it often erodes the cement paste and loose sand grains. More importantly, the water trapped in cracks may lead to the wedging actions of hydraulic fracturing, i.e., the trapped water in the crack can cause hydraulic fracture and splitting of the concrete under the instantaneous hydrostatic pressure. Laboratory tests show that submersion of concrete leads to a substantial reduction of fatigue life due to the pumping and wedging of water. Because submersion of concrete can accelerate the fatigue failure under cyclic loading, fatigue failure criteria for submerged concrete are more restrictive than for concrete sections above the water.

14-3 Stress Limitation Control

The stress limitation control is a simplified design method that evaluates the fatigue strength on the basis of a set of allowable stress criteria for concrete, reinforcing, and prestressing steel. The design method is based upon the assumption that a structural element is safe against fatigue failure if certain stress conditions are satisfied.

The fatigue strength requirements shall be met by limiting the allowable stresses in steel and concrete as follows:

a. Maximum stress range in reinforcing and prestressing steel is less than 20,000 psi (140 MPa). If reinforcement is bent or welded, the maximum allowable stress is 10,000 psi. (70 MPa).

b. No membrane tensile stress in concrete.

c. Maximum flexural tensile stress in concrete is less than 200 psi (1.4 MPa).

d. Maximum compressive stress in concrete is less than $0.5f_c'$.

e. If maximum shear exceeds the allowable shear stress in concrete, or if the cyclic excursions exceed 50 percent allowable shear in concrete, stirrups should be designed without consideration of strength contribution from concrete. Calculation of the allowable shear in concrete may account for the favorable effect of prestressing.

If any one of the above stress values in steel or concrete is exceeded, a more detailed fatigue analysis based on the principle of cumulative damage should then be carried out to verify the fatigue strengths of concrete, reinforcement, and bond between concrete and steel, as described in Section 14-4 below.

14-4 Comprehensive Fatigue Analysis

The comprehensive fatigue analysis is based upon the theory of linear cumulative damage. The design method requires that the long-term distribution of wind, wave, and current forces and impact loads upon a hydraulic structure be established and subdivided into a number of load blocks in the form of a histogram. Then, the dynamic response of the structure is analyzed for each wave block, including appropriate dynamic amplification.

On the basis of the analysis results, total cumulative fatigue damage under the entire variable loading spectrum can be determined in accordance with the linear theory of cumulative damage, i.e., Miner-Palmgren hypothesis together with the Wöhler *S*-log *N* curves. The basic assumption of the theory implies that the long-term distribution of stress range can be represented by a stress histogram consisting of a number of constant amplitude stress range blocks, each with the appropriate number of stress repetitions:

$$\sum_{i=1}^{j} \frac{n_i}{N_i} \leq \eta \qquad (14\text{-}1)$$

where η is Minor's sum and typically ranges from 0.2 to 0.5 for concrete structures, depending on the exposure condition.

The above theory can be applied to evaluate fatigue strengths of concrete, reinforcing and prestressing steel, and bond between the steel and concrete under an assumed cyclic loading histogram, provided that appropriate *S*-log *N* curves are applied.

The fatigue strengths of concrete and steel, as well as the bond strength under a single amplitude of loading, are usually determined in accordance the Wöhler *S*-log *N* design curves. The *S*-log *N* curves reflect dependence of the fatigue strength on the stress range, material properties, and environmental conditions. The curves can be generally represented as

$$\log N = C_1\left(\frac{1-\sigma_{\max}/f_{rd}}{1-\sigma_{\min}/f_{rd}}\right) \qquad (14\text{-}2)$$

where

C_1 = a factor dependent on the environmental conditions (For example, C_1 is 1.2 for concrete in air, and 0.8 for submerged concrete subjected to compression/tension stress reversal.)

σ_{max}, σ_{min} = largest and smallest compressive stress calculated as the mean stress within each stress block. (For tensile stress, $\sigma_{min} = 0$.)

f_{rd} = design strength corresponding to the referenced failure mode

The fatigue limit state of a prestressed concrete member should be evaluated in terms of the flexural strength, shear strength, and bond strength.

a. *Flexural strength.* On the basis of fatigue characteristics of the basic materials, calculations of the flexural fatigue strength of concrete members are straightforward. In determination of the critical fatigue stresses from the *S*-log *N* curves, it is important to distinguish cracked concrete section from uncracked concrete section. In general, cycling of reinforced concrete into tension and cracking will lead to a progressive degradation in the effective prestress and stiffness, which in turn results in wider cracks and higher steel stresses. One of the most effective ways to increase the fatigue strength is the use of prestressing to limit tensile stress level in concrete and control occurrence of cracks. The current fatigue design practice is to prestress concrete so that the concrete stresses rarely cycle into the tension range. If the effective prestress is sufficient to ensure structural members within the state of precompression under the service loading condition, the fatigue failure will not occur.

b. *Shear strength.* Reinforced concrete beams, with or without web reinforcement, can have shear failure under cyclic loading at as little as 30 to 40 percent of the static strength. For prestressed concrete beams, shear failure under cyclic loading can be controlled by imposing prestress in concrete. The fatigue strength for web-shear cracking can be evaluated by limiting the principal tension stress in accordance with the modified Goodman diagram (Figure 14.1). The fatigue strength for flexure-shear cracking can be evaluated by limiting the extreme fiber tension stress. As the magnitude of shear stress and number of the load cycles increase, the possibility of web-shear cracking occurring prior to flexure-shear cracking increases. As inclined cracks initiate, the web reinforcement will carry all the shear stresses after a number of loading cycles. The fatigue life of prestressed concrete beams after diagonal cracking should be based on predicted stress ranges in the stirrups, using the truss analogy and assuming all the shear stresses carried by the web reinforcement.

In thin-wall panels subjected to cyclic or repeated loads, the cyclic degradation is often magnified under extreme cyclic reversing membrane shear force. The reversing shear can produce a double pattern of diagonal cracks, oriented at an angle to the grid pattern of reinforcement. The cyclic membrane shear can be viewed as a case of alternating cyclic diagonal tension and compression. When only conventional reinforcement is used, the concrete will crack under moderate to high shear forces. Then, the reinforcement is expected to carry all the shear tension force across the cracks. The stress range must be kept low to minimize the crack width and cumulative fatigue damage. For the typical diagonal crack pattern, crack widths are

enlarged as compared with cracks normal to the reinforcement at the same level of steel stress, because the reinforcement layout is not effective in controlling this type of inclined cracking. Under high-intensity loading, displacement along the cracks may produce abrasion of the concrete surfaces and a rapid reduction in shear stiffness.

c. *Bond strength.* Bond deterioration between steel and concrete is known to be a significant factor in fatigue failure of concrete structures under low-cycle high-amplitude cyclic loading. Although quantitative information on the effects is still lacking, it has been found that deterioration of bond strength under cyclic loadings is influenced by several factors, including depth of concrete cover, confinement of concrete around the steel, cyclic tensile stress range in steel, diameter and deformation of steel reinforcement, water pumping through cracks for submerged concrete, and concrete strength.

Fatigue tests show that fatigue failure of reinforced concrete was often preceded by cracking and loss of bond in association with splitting of concrete and yielding of reinforcing steel. The bond failure usually exhibits the two distinctive modes. The first failure mode is related to tensile splitting failure of concrete, which is not fully confined. The second mode is pullout failure, which is related to shear failure of the concrete. Tensile splitting failure usually exhibits much lower bond strength than that of pullout failure. This indicates that the ultimate bond strength is highly dependent on the confinement and geometry of the concrete, as well as the loading pattern. In general, the bond strength under 10^6 cycles of loads will be approximately 60 percent of the static strength. Where inclined cracks occur under repeated loading or there are significant splitting effects because of inadequate concrete cover or inadequate lateral reinforcement around the bars, bond strength can decrease to 40 percent of the static strength under cyclic loading. The lateral confinement of longitudinal steel bars can significantly enhance concrete bond strength. When concrete cover is not adequate to prevent splitting bond failure, sufficient lateral steel confinement should be provided to prevent premature bond failure under repeated cyclic loads.

For high-amplitude loading, the local bond stress builds up to a maximum and then deteriorates for increasing peak slips. The rate of deterioration depends on the previous maximum local bond slip. The greater the previous bond slip, the greater the reduction in bond stress. Reversals of loading on the steel bars increase the rate of bond deterioration by approximately 50 percent.

15 Load Combinations for Thin Walls

Notation

$\mathbf{f_{c2}}$ = compressive strength of the concrete, considering the potential reduction of the strength in the state of bi-axial compression/tension

$\mathbf{F_x}$ **or** $\mathbf{F_y}$ = design force for reinforcement along x-axis or y-axis

h = thickness of the thin-wall element

N = axial force in an equivalent beam strip

$\mathbf{N_x}$ **or** $\mathbf{N_y}$ = membrane tension or compression forces

$\mathbf{N_{xy}}$ = membrane shear force

$\mathbf{M_x}$ **or** $\mathbf{M_y}$ = bending moments

$\mathbf{T_{xy}}$ = torsion

$\mathbf{V_{xz}}$ **or** $\mathbf{V_{yz}}$ = out-of-plane transverse shear forces

V = shear force in an equivalent beam strip

$\boldsymbol{\theta}$ = angle of the principal membrane stress to the x-axis

$\boldsymbol{\alpha}$ = angle between the axis of an equivalent beam strip and the x-axis

$\boldsymbol{\sigma_c}$ = membrane compressive stress in concrete

15-1 General

In prefabricated construction, a typical P/C module consists mainly of plates and shells and is subjected to various complex loads during different load stages. Therefore, structural analysis is commonly performed with the finite element method. For all the critical load combinations, sectional forces (axial loads, bending moments, and shears) are calculated at various critical locations of the structure. At every location, the sectional strength is checked against the loading demand. The loading demand on a typical plate or shell can be expressed in terms of eight stress resultants (see Figure 15.1), that is, three membrane forces (N_x, N_y, and N_{xy}), three bending moments (M_x, M_y, and T_{xy}), and two out-of-plane transverse shear forces (V_{xz} and V_{yz}).

The design of reinforced concrete plate/shell under various load combinations has been one of the most controversial subjects. During the last several decades, various theories and design methods have been proposed. Most design codes do not specify any specific analysis method. Currently, there are three analysis methods for sectional strength design of plate/shell elements:

(1) Principle of superposition of component resistance.

(2) Rational analytical models using a failure criterion for the multi-axial state of stress in the concrete, e.g., the Modified Compression Field Theory.

(3) Variable strut-tie model.

The traditional design has been based upon the principle of superposition. It is a simplified equilibrium approach that decouples the effects of transverse shear from those of membrane forces and moments. The design of plate/shell elements subjected to the three membrane forces and three bending moments is a generalization of the strain compatibility approach. The design for the transverse shear is based upon the equivalent beam assumption and empirical rules.

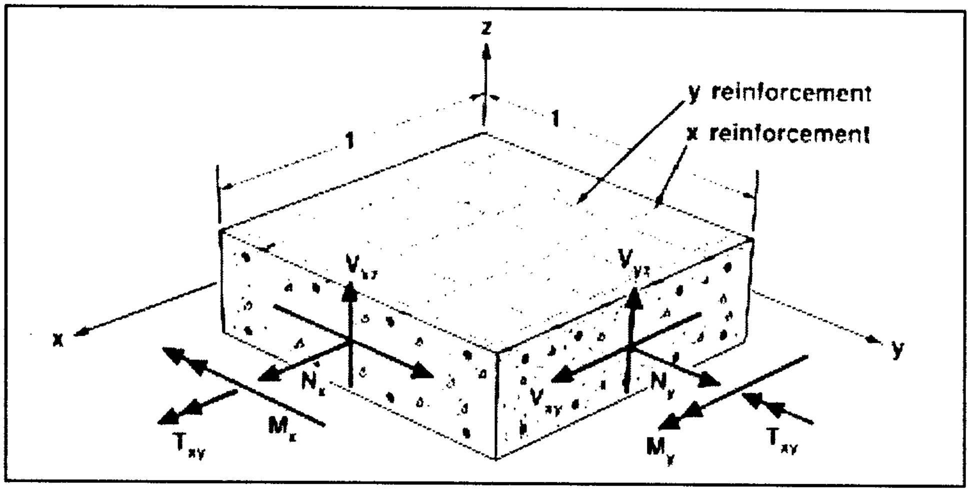


Figure 15.1. Sectional forces in a thin-wall element

The multi-axial stress field analysis method is intended to address some deficiencies with the traditional design method. The method finds a compatible strain-stress field that balances the sectional forces by an iterative process. Equilibrium is then established at the crack surface to determine the ultimate strength, ductility, and failure mode of each individual element. Due to its analytical complexity, the stress field analysis typically relies on computer programs.

The strut-tie model is a very effective tool for investigating the discontinuity zone. It can also be used, although less frequently, to analyze a large portion of the structure.

The structural strength design of plate/shell elements to a great extent depends on the primary types of loading to which the shell elements are subjected. In general, the design of plate/shell elements may be divided as (1) shell subjected to membrane forces only, (2) shell subjected to membrane forces and bending, and (3) shell subjected to membrane forces, bending, and transverse shear.

In the following, only the simplified equilibrium approach and the stress field method will be discussed.

15-2 Plate Subject to Membrane Forces Only

When plates and shells in P/C segments are primarily subjected to membrane forces (N_x, N_y, N_{xy}), analysis can be conducted considering only the in-plane stress and strains, assuming the stresses and strains are constant through the thickness. A simplified equilibrium approach is allowed to design membrane plate/shell elements without considerations of the tensile strength of concrete and deformation. This approach is simple for hand-calculation and generally satisfactory for design of membrane elements.

The reinforcement along the x-axis and y-axis is designed to carry two sets of forces, F_x and F_y, respectively. The forces F_x and F_y are to be determined from equilibrium as follows:

$$F_x = N_x + |N_{xy}| \cot\theta \tag{15-1}$$

$$F_y = N_y + |N_{xy}| \tan\theta \tag{15-2}$$

In addition, the compressive strength of concrete in the principal direction at an angle θ to the x-axis should be checked against compressive failure as follows:

$$\sigma_c = \frac{|N_{xy}|}{h \sin\theta \cos\theta} \leq f_{c2} \tag{15-3}$$

where f_{c2} is compressive strength of the concrete, considering the potential reduction of the strength in the state of bi-axial compression/tension, and h is the thickness of the element.

The signs of the membrane forces and stress σ_c are shown Figure 15.2. When the membrane force N_x is compressive (negative in value) and exceeds $|N_{xy}| \cot\theta$, F_x is negative and only minimum reinforcement is required in the x-direction. Then, the reinforcement in the y-direction should be designed for F_y as follows:

$$F_y = N_y + \frac{N_{xy}^{\ 2}}{|N_x|} \tag{15-4}$$

When both N_x and N_y forces are compressive and $N_x N_y \geq N_{xy}^2$, only minimum reinforcement will be required.

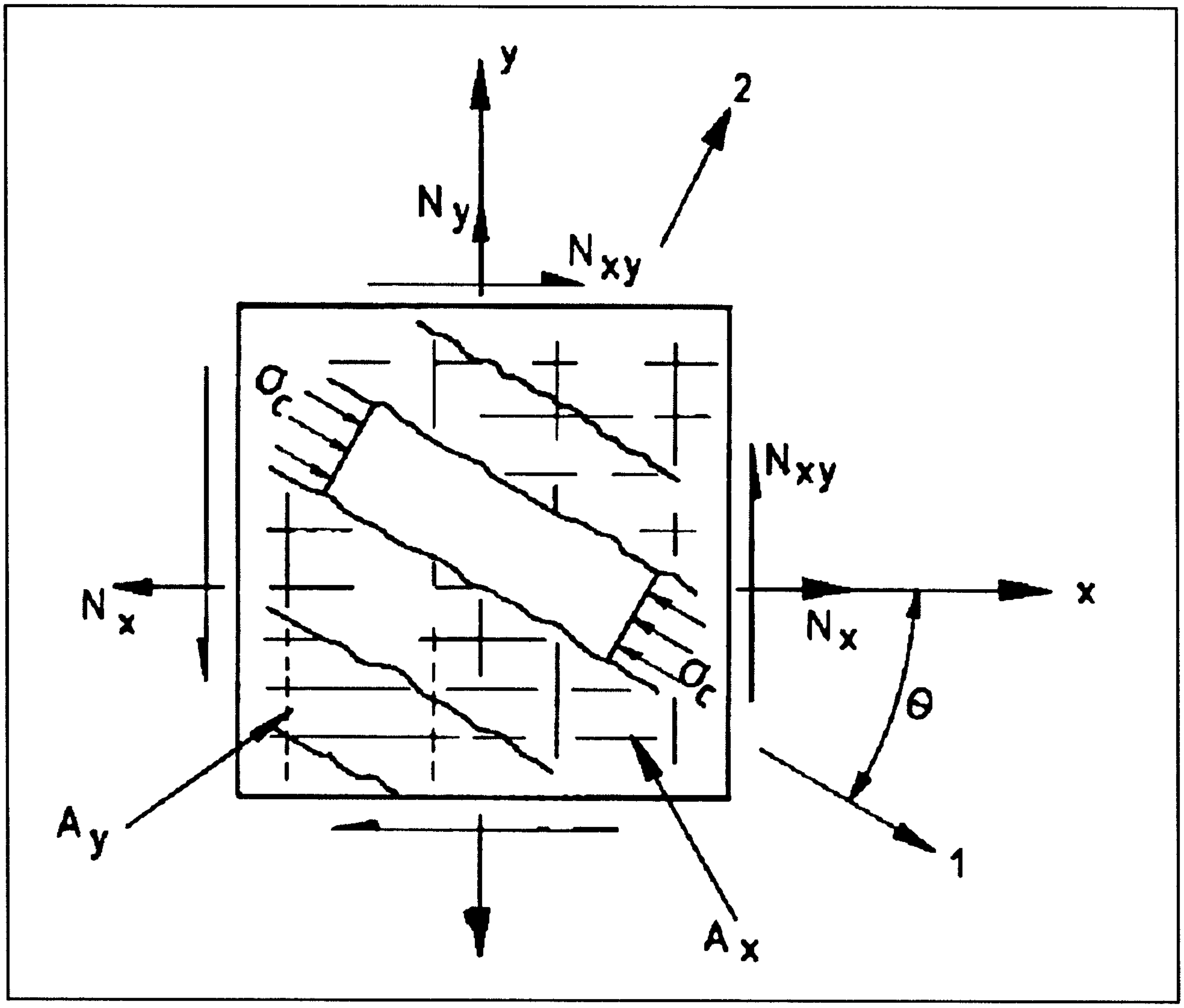


Figure 15.2. Sectional forces and symbols of a membrane plate/shell element

15-3 Plates Subject to Membrane Forces and Bending

When a plate/shell element is subjected to both membrane forces (N_x, N_y, N_{xy}) and bending moments (M_x, M_y, T_{xy}), analysis can be conducted considering only the in-plane stress and strains, assuming the stresses and strains vary linearly through the thickness.

Therefore, the simplified equilibrium method can be used in analysis in a similar manner as that for membrane plate/shell elements. The basic procedure is to divide the shell element into a sandwich consisting of two outer layers and a

central zone. The membrane force and moments are resolved into statically equivalent "membrane" forces on the out layers, as shown in Figure 15.3. Then, equilibrium can be established to calculate the reinforcement in the top and bottom layers.

As an alternative to the equilibrium method, some computer programs have been developed to calculate the stresses, strains, and sectional resistance using the layered shell approach and a sophisticated Modified Compression Field Theory (Fiskvatin and Grosch 1982, Kirschner and Collins 1986). The computer analytical model subdivides the plate/shell element into many layers, as shown in Figure 15.4. Within each layer, only the in-plane stresses and strains are considered, and the Modified Compression Field Theory is applied. The strains are calculated from the sectional forces by an iterative analysis. Then, stresses are calculated according to the multi-axial stress-strain relationships. It is assumed that a valid strain state is one in which the stress resultants found by integrating the resulting stresses balance the applied sectional forces.

Figure 15.5 illustrates the experimental results and analytical predictions of response of shell elements subjected to various combinations of membrane shears and bending moments. It is apparent that the flexural bending strength of the shell element is strongly coupled with the in-place shear forces.

This interactive response of shell element under the combined load actions is especially important for very large prefabricated concrete segments for in-the-wet construction. Unlike building structures where significant membrane shear and bending moment rarely occur at the same location, plate/shell elements in the precast modules are frequently subjected to large membrane tension, membrane shear, and out-of-plane moments during construction. For a large bending moment, high membrane shears may initiate cracking in shell, and also substantially reduce the ultimate strength.

15-4 Plates Subject to Membrane Forces, Shear, and Bending

In the design of P/C modules used for in-the-wet construction of navigation structures, significant shear forces and bending moments sometimes have to be transferred at intersections between structural elements, resulting in very high membrane forces, moments, and shears in the adjoining members. Adequate strengths at these intersections and the adjoining members are critical to the integrity of the structure.

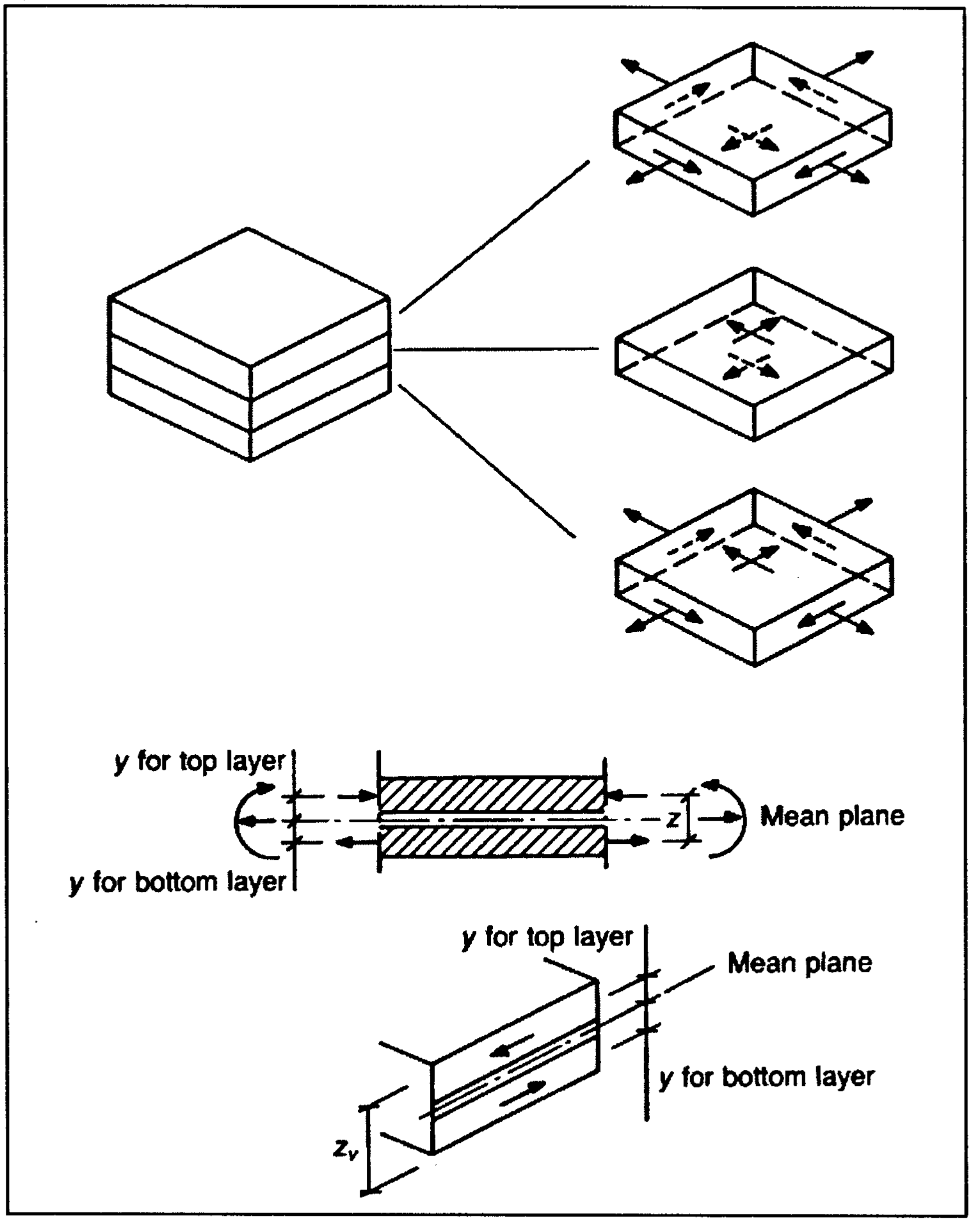


Figure 15.3. Equilibrium approach–sectional forces are resolved into membrane forces in sandwiched layers

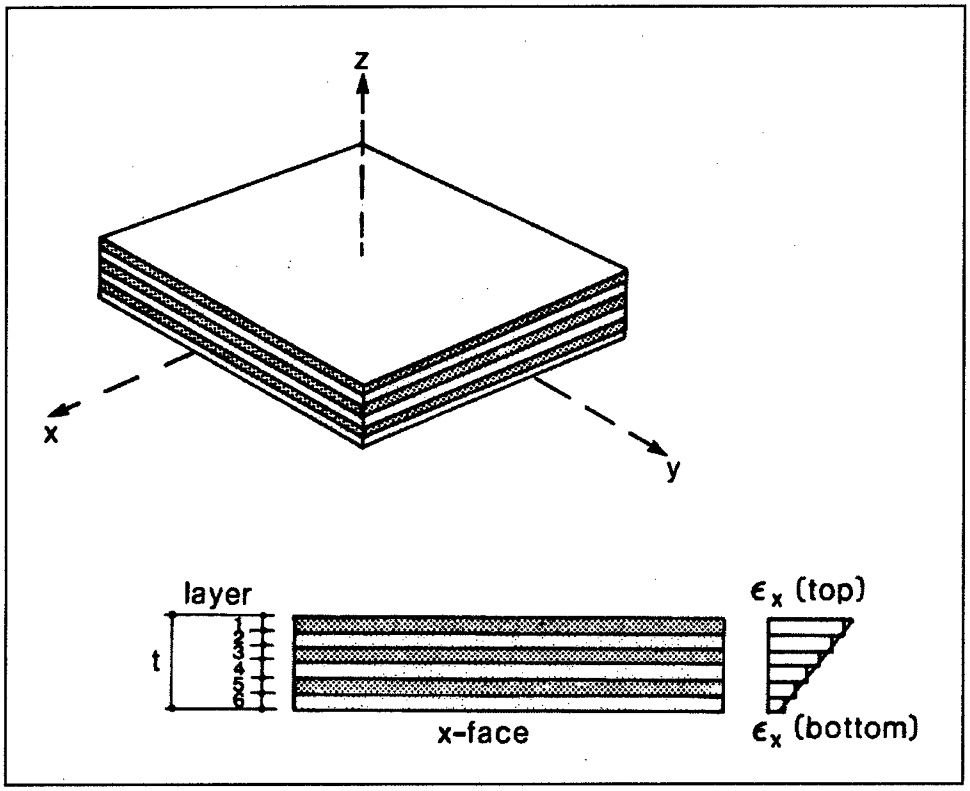


Figure 15.4. Layered shell element

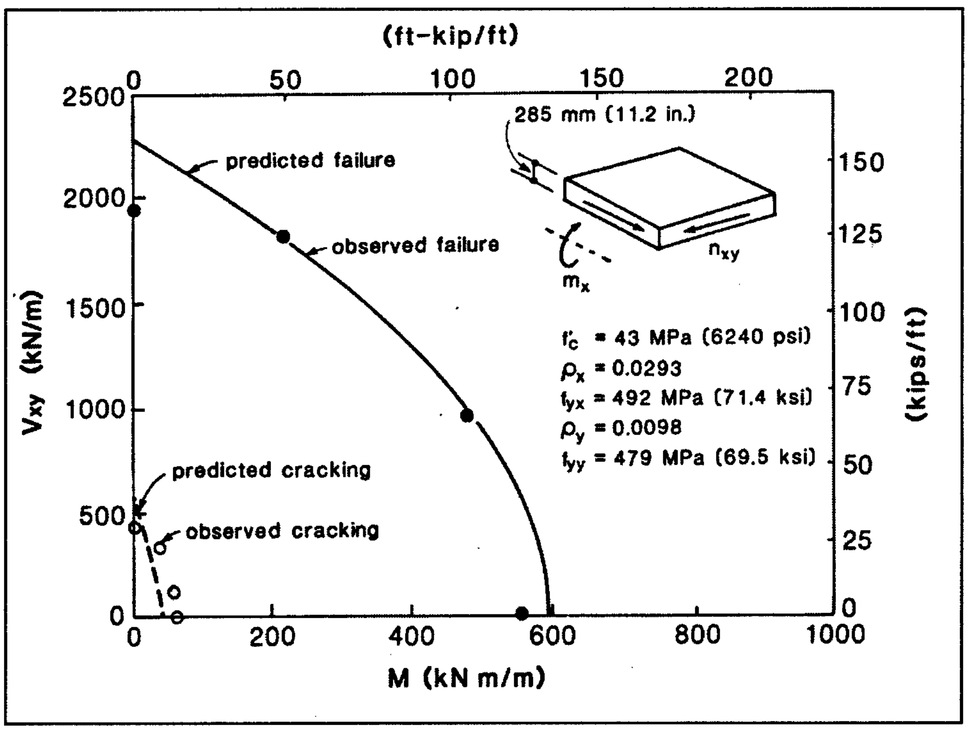


Figure 15.5. Shear-moment interaction diagram

Therefore, stringent requirements on shear strength design should be imposed. The most common shear design requirements are as follows:

- The bending moment and deformation loads must be properly considered in shear capacity calculation.
- In slabs and walls, shear transfer should be checked in all directions on the basis of elastic analysis, unless a thorough study documenting redistribution of forces confirms the validity of an alternative approach.
- In sections having a combination of high compressive stresses and high shear forces, the main reinforcement should be confined with through-thickness stirrups in an amount not less than 0.4 percent of the concrete area.
- All load-bearing embedments must be properly anchored into the concrete with reinforcement (additional stirrups) to transfer the tension force. This requirement addresses the potential pull-out or punching-through type of failure where only part of the thickness of the structural element is activated.
- Tension forces perpendicular to the shell plane, if any, must be taken by extra reinforcement properly anchored in the concrete.
- All the prestressing tendons should be confined between layers of transverse reinforcing steel in slabs and walls. Curved prestressing tendons shall be adequately confined by lateral reinforcement.

The traditional shear design practice has been based on the principle of superposition of component resistances, or the so-called "equivalent beam" method. In this method, evaluation of the transverse shear resistance is separated from evaluation of the flexural strength and in-plane shear strength. The evaluation of the sectional shear strength of shell elements has traditionally been based upon empirical procedures developed for one-dimensional members.

The shear design procedure on the basis of the superposition principle is to take an equivalent beam strip in a shell element, as shown in Figure 15.6, and then calculate the transverse shear force and the axial force on unit width of the beam strip as follows:

$$V = V_{xz} \cos\alpha + V_{yz} \sin\alpha \qquad (15\text{-}5)$$

$$N = N_x \cos^2\alpha + N_y \sin^2\alpha + 2N_{xy} \sin\alpha\cos\alpha \qquad (15\text{-}6)$$

where α is the angle between the axis of the beam strip and the x-axis.

The shear and axial force so obtained can be applied directly to the conventional beam design equations, such as those stipulated by EM 1110-2-2104, AASHTO Specification, or ACI 318 Building Code.

For one-dimensional elements (beams), all the axial loads, the transverse shear, and the longitudinal reinforcement are parallel. For shell and plate, the loading condition can be much more complex. For example, considering an equivalent beam strip taken out of a shell element (Figure 15.6), the beam strip may be subjected to the principal transverse shear on the unit width without axial tension. But the "beam" would also be subjected to axial tension on its "side faces" across the width. The influence of the tension on the shear strength cannot be accounted for in the equivalent beam design procedure.

Past experience has proven that the equivalent beam method is valid for shear forces combined with moderate bending moments and axial forces. However, when high bending moments and high axial forces occur simultaneously in shell elements, the equivalent beam method may give inaccurate results.

A close examination of the equivalent beam approach reveals several drawbacks with the method. To apply the beam shear equations to the equivalent beam strip (see Figure 15.6), the forces on the side faces of each beam strip are neglected. The influence of the orientation of the reinforcement is also neglected. As a result, the method cannot accurately predict the influences of membrane shear and axial force on the transverse shear strength of the structures.

When a plate/shell element is subjected to a combination of high shear, bending moment, and axial forces, it is justifiable to use more refined shear design methods. For example, the CSA (1994) S474 and the DNV (Det Norske Veritas) codes recommend the use of the analytical models based upon the Modified Compression Field Theory.

The analytical model based on the Modified Compression Field Theory has been incorporated into computer programs (Collins 1989). Figure 15.7 illustrates the observed test results and predicted responses of a number of shell elements subjected to combined membrane shear, transverse shear, and bending moment. The analysis results of the stress field model are compared with those of the equivalent beam method in the figure. It is apparent that the equivalent beam method fails to predict the interaction between the membrane shear and transverse shear.

Tests also show that the equivalent beam method may become considerably unconservative when very high membrane compression coexists with significant transverse shear, and may become excessively conservative when high membrane tension exists.

It is noted that the ASSHTO Bridge Specification (LRFD) has formally adopted the shear design method for beams and girders based on the modified compression field theory. ACI 318 Committee is in the process of adopting the stress field method in lieu of the traditional empirical shear design procedure.

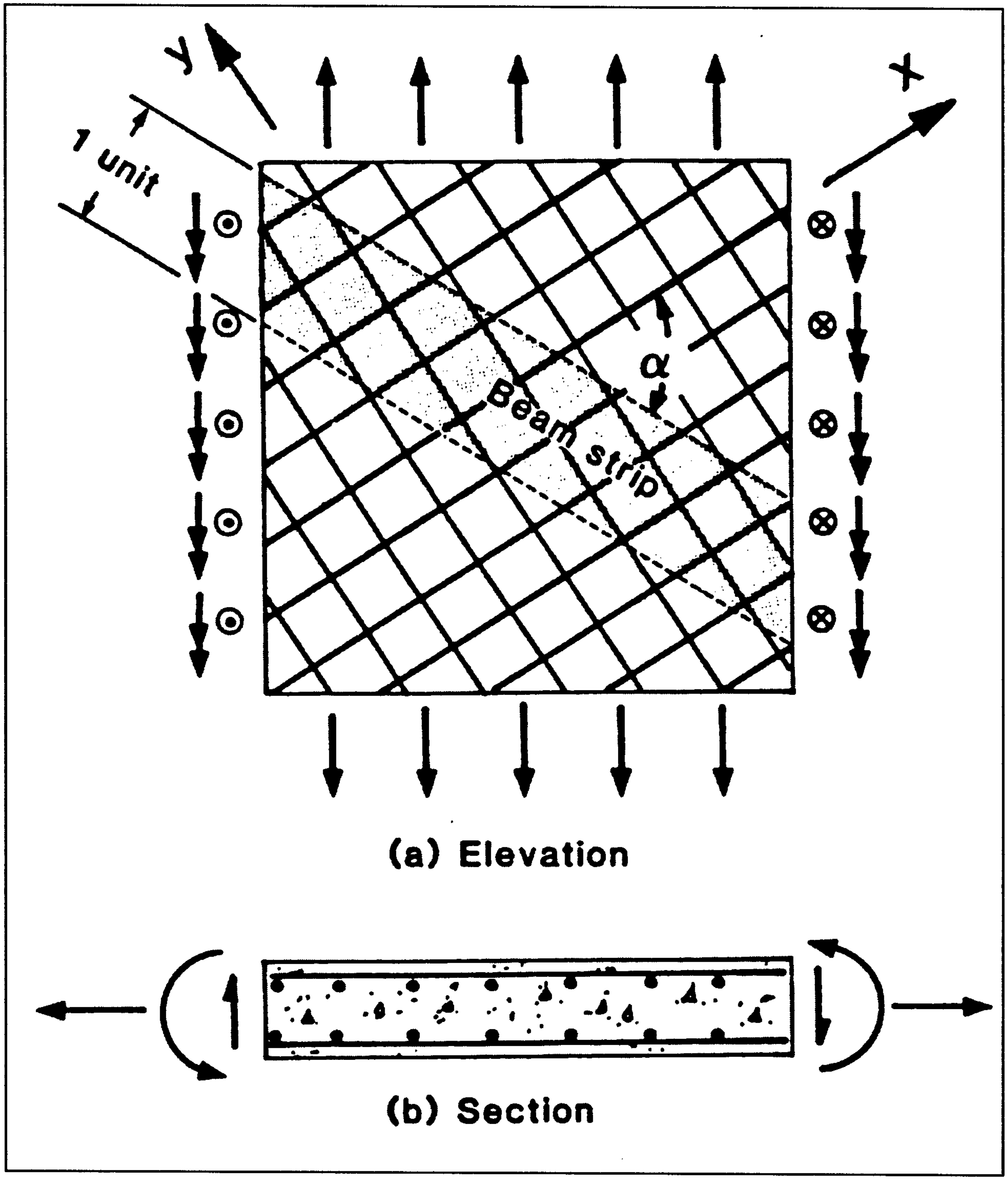


Figure 15.6. Equivalent beam model as applied to shell elements

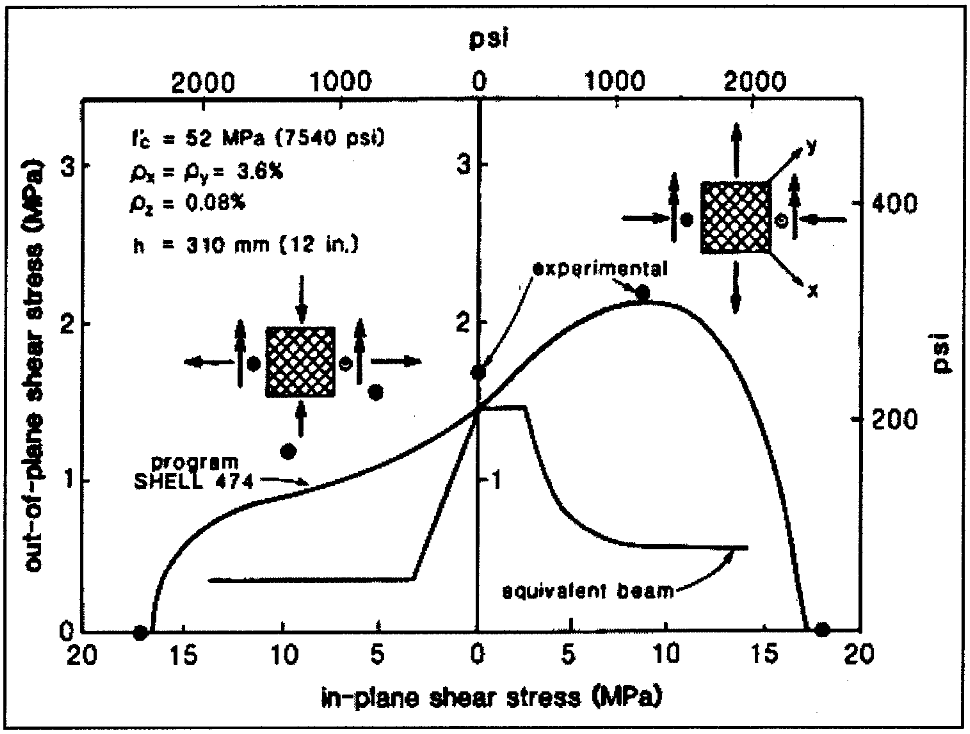


Figure 15.7. Interaction among membrane shear,transverse shear, and moment

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Appendix A
Design Philosophy

Notation

T = deformation loads

The deterministic design requirements cited in this report are founded upon probability principles. The objective of the deterministic design requirements is to establish a design methodology that defines limit state margins for the structural response to all anticipated loads, deformations, and environmental conditions for the duration of its design life, with adequate safety. Acceptable safety levels are a function of the consequence of limit state failures. Target annual reliabilities are intended to ensure that the limit state failure probabilities (given in the table below) are not exceeded.

Table A.1
Target Annual Probabilities of Limit State Exceedance

Consequence of Limit State Failure	Target Annual Probability of Limit State Exceedance
High risk to life, environment, or navigation*	1×10^{-5}
Low risk to life, environment, or navigation	1×10^{-3}
Impaired function	1×10^{-1}
* Where high risk to life, environment, or navigation exists, active measures shall be taken to mitigate such risks.	

The deterministic methodologies presented in this report have been simplified for practical applications as compared to fully probabilistic methodologies. The ultimate strength design methodology adopted in this report uses basic load factors, hydraulic factors, and strength reduction factors. For serviceability limit states, including those for construction stage serviceability cases, this report adopts the use of the Working Stress Design methodology. Deviations from these guidelines may be allowed by Corps Headquarters, provided that adequate load, deformation, or environmental data and/or member limit state capacity test results are provided.

When deviating from the guidelines for strength design, new basic load factors may be developed such that, when taken in combination with the hydraulic factors and strength reduction factors, they result in target annual probabilities of limit state exceedance cited in Table A.1. The basic load factors

should be used together with specified external loads that have target annual exceedance probabilities, as shown in Table A.2.

The target exceedance probabilities for the occurrence of disruptive construction events shall typically be less than 5×10^{-2} for the exposure period during construction; however, the target exceedance probability of occurrence for specific significant events should take into account the consequence of exceedance.

Table A.2
Target Annual Exceedance Probabilities for External Loads

Specified External Load (During Operation)	Target Annual Exceedance Probability
Usual environmental	10^{-1}
Unusual environmental	10^{-2}
Extreme environmental	10^{-4} to 10^{-3}
Accidental	10^{-4} to 10^{-3}

The strength design methodology shall prevent

- Loss of equilibrium of the structure, or component of the structure (e.g., overturning, capsizing, sliding, or sinking in water or liquefied soil).
- Formation of a collapse mechanism.
- Loss of load-carrying capacity of structural elements due to the material strength being exceeded, buckling, fracture, fatigue, fire, or excessive deformation.
- Loss of stability (e.g., P-Delta effect, flutter, etc.).
- Excessive plastic or creep deformation.

For strength design, deformation loads, T, shall have an appropriate load factor, and shall include the cumulative effects of

- Temperature changes and gradients.
- Creep and shrinkage.
- Differential settlement, and foundation stiffness variations.
- Prestressing.

The serviceability design shall prevent

- Excessive cracking, or local damage (e.g., scour/erosion, splitting, spalling, local yielding of steel reinforcement, or slippage of connections).
- Unacceptable deformations, or rotations.
- Corrosion or reinforcement, or deterioration of concrete.
- Excessive leakage, or excessive maintenance.
- Undesirable motions, or vibrations.

Appendix B
Lessons Learned at Braddock Dam Construction

The construction of the new gated dam at Braddock Locks and Dam on the Monongahela River just south of Pittsburgh began in late 1999 and is expected to be completed in late 2002 or early 2003. This project utilized precast thin-wall panels for the following dam features:

- Two large, roughly 330-ft-long by 105-ft-wide float-in segments for the base of the new dam. Each segment has roughly 200 precast panels fabricated into a multicellular structure at the Leetsdale Industrial Park on the Ohio River. The panels were fabricated inside industrial buildings on site and stored in adjacent yards prior to erection in the two-level casting basin. (The panels are 255 mm (10 in.) and 305 mm (12 in.) thick and vary in dimensions.) Photo B1 shows panel casting; Photo B2, yard storage; Photo B3, panel erection; and Photo B4, segment construction.

- Thirty-one precast panels (380 mm (15 in.) thick; 9 m (30 ft, 6 in.) long; 6 m (20 ft) wide) for the tailrace structure. These panels have not yet been fabricated, but the contractor currently plans to build them on site.

- Five piers above elevation 726 will have 20-cm (8-in.)-thick precast panels as stay-in-place forms for in-the-dry pier construction. These panels also have not yet been fabricated, and may be contracted out to a precast manufacturer.

This project is being constructed for the U.S. Army Engineer District, Pittsburgh. Team members at the Pittsburgh District have been monitoring the results of innovative uses of precast panels for this facility. For complete explanations of these lessons, contact the Pittsburgh District directly. Summarized below are the lessons learned to date from this process.

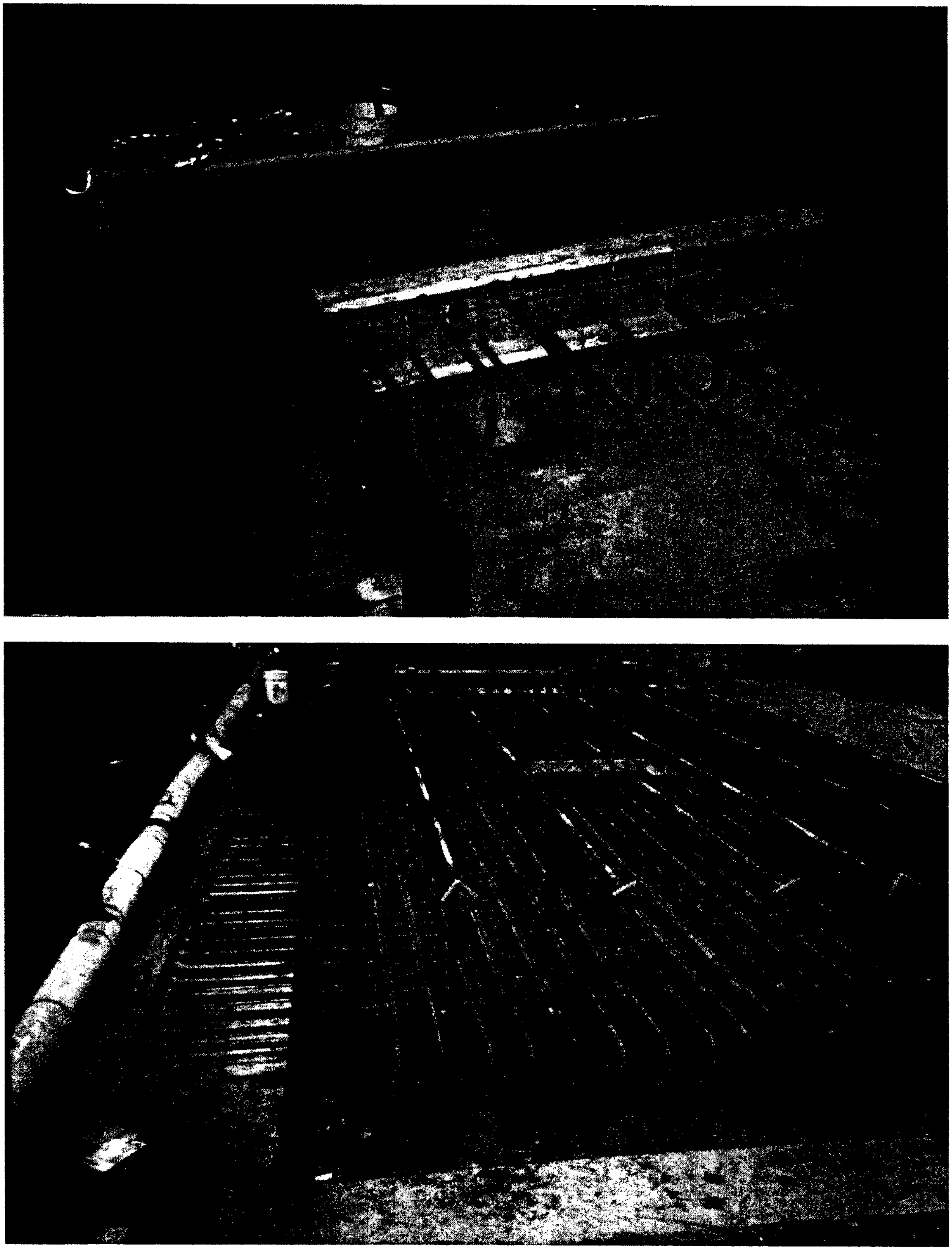


Photo B1. Panel casting

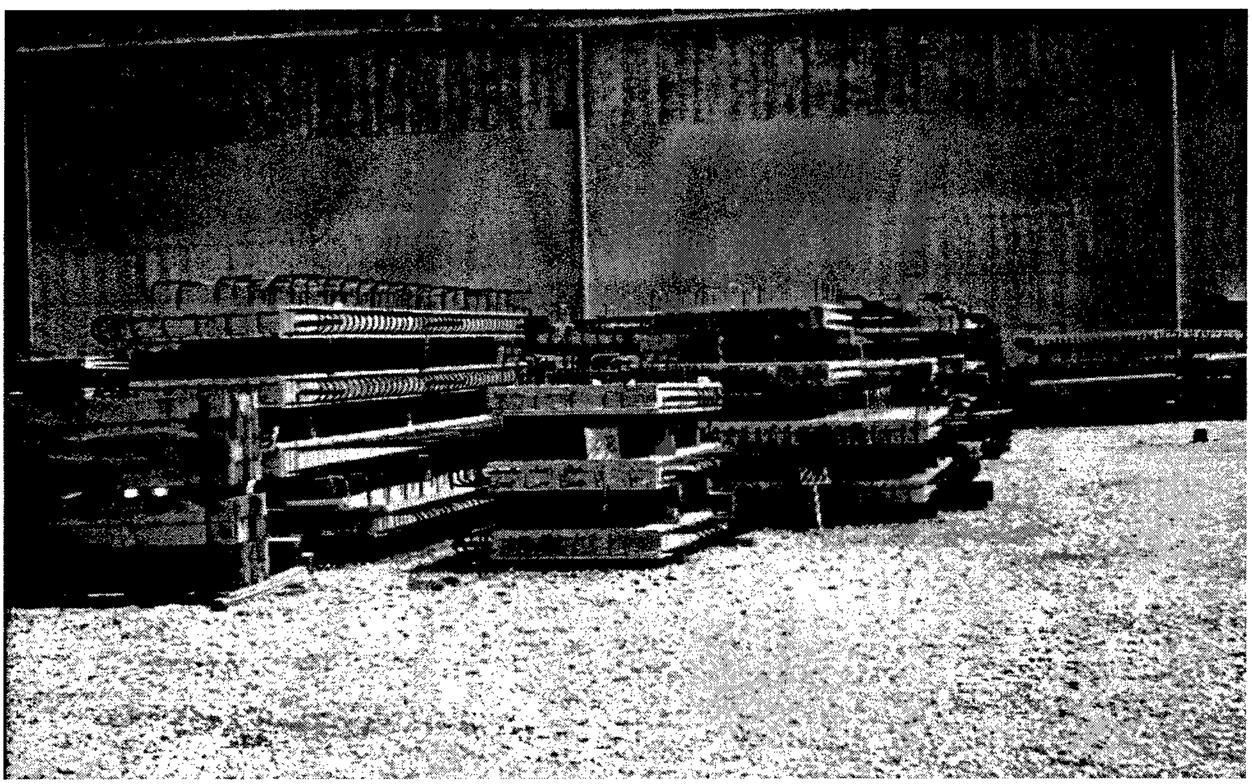


Photo B2. Yard storage of panels

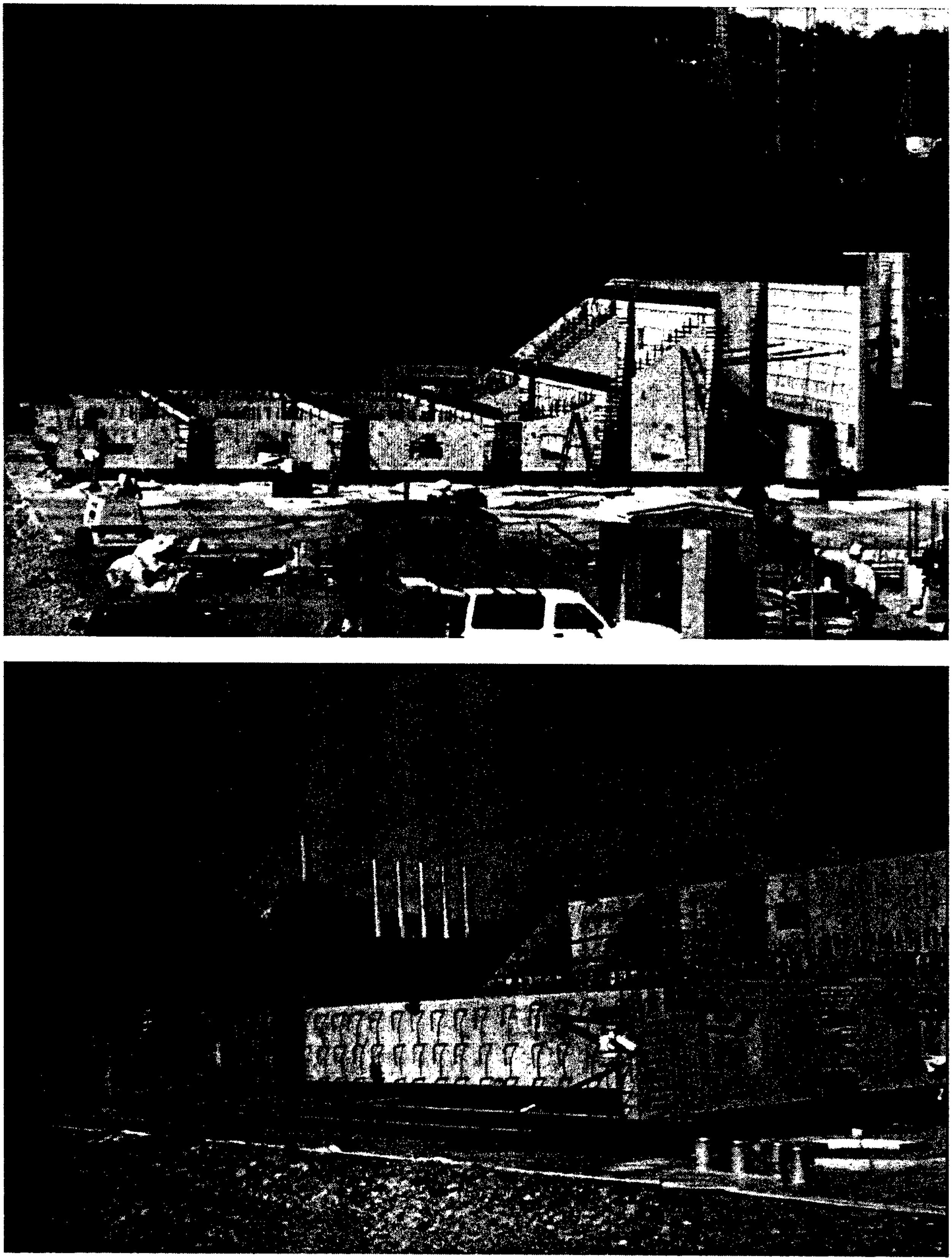


Photo B3. Panel erection

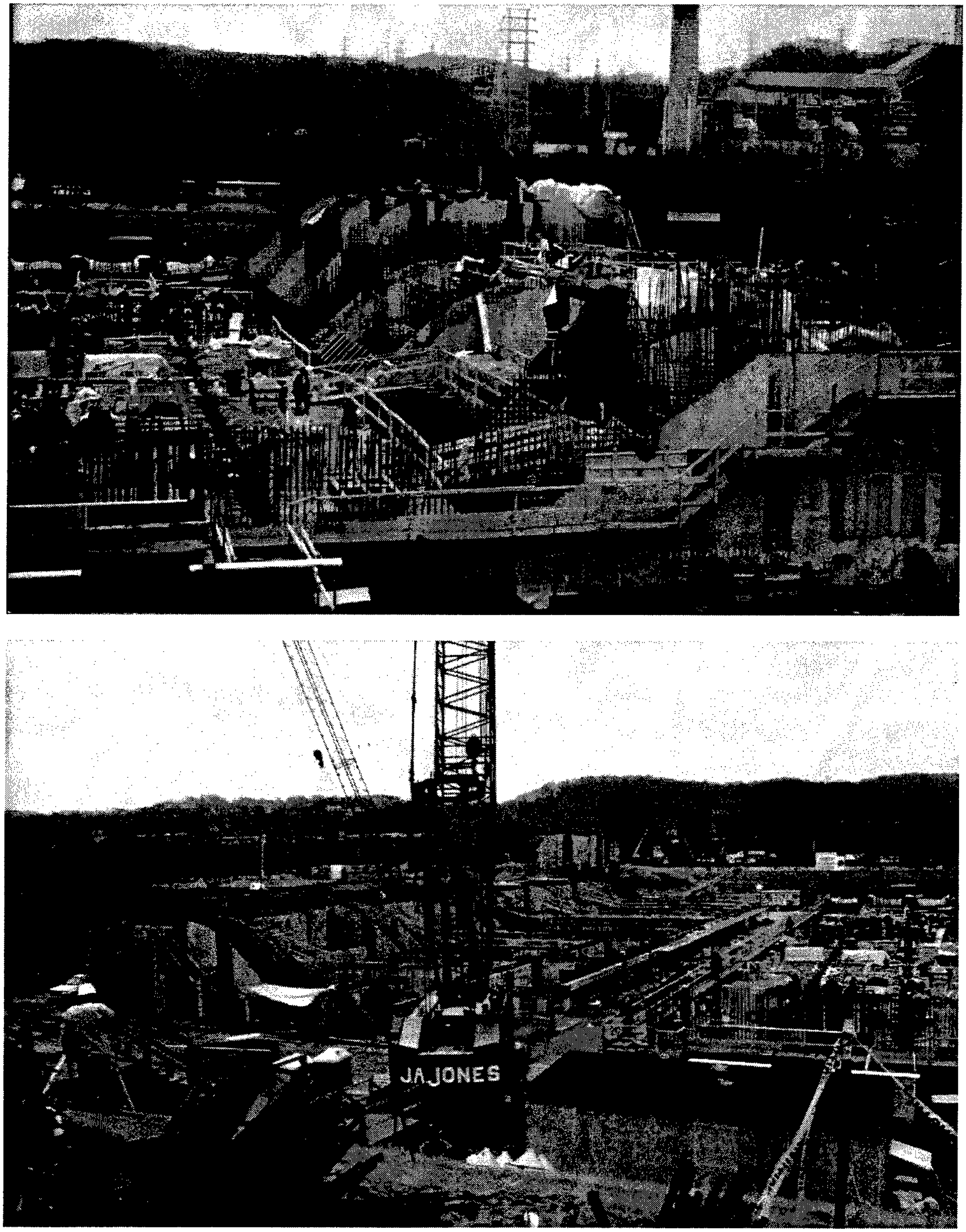


Photo B4. Construction of segments

Panel Fabrication

a. Numerous surface finish techniques were tested to achieve a roughened surface on internal faces of segment panels, thereby providing mechanical interlock with infill concrete. The most effective techniques were the use of welded wire fabric covered by plastic sheeting for bottom cast surfaces and simply leaving a vibrated finish on top surfaces (see Photo B5).

b. For the 438 precast panels, six panel shapes were built in 97 sizes. This made the use of standard forms difficult and reduced the efficiency of precasting operations. Keeping the number of shapes and sizes to the absolute minimum, and adjusting individual panels after erection with cast-in-place extensions, will help to reduce costs.

c. Designers should allow contractors the option to cast all panels in place. If precasting is selected, assume that the contractor will fabricate panels locally, not in a commercial precasting facility. This leads to consideration of a commercial ready-mix plant in lieu of an on-site batch plant. The Braddock Dam project allowed a Value Engineering proposal to use a local ready-mix concrete plant, which has considerations for material control, quality control staffing, concrete availability, and transport times.

d. Field steam curing of precast panels was successfully used to reduce curing time. Both concrete and ambient temperatures were recorded to develop an effective steam cycle with an adequate ramp-down period prior to lifting and transport to storage areas.

e. The large amount of rebar in segment panels required increased support chairs to keep heavy mats of reinforcing steel from deflecting on the casting bed and resulting in reduced cover on bottom cast surfaces and increased cover on top surfaces (Photo B6).

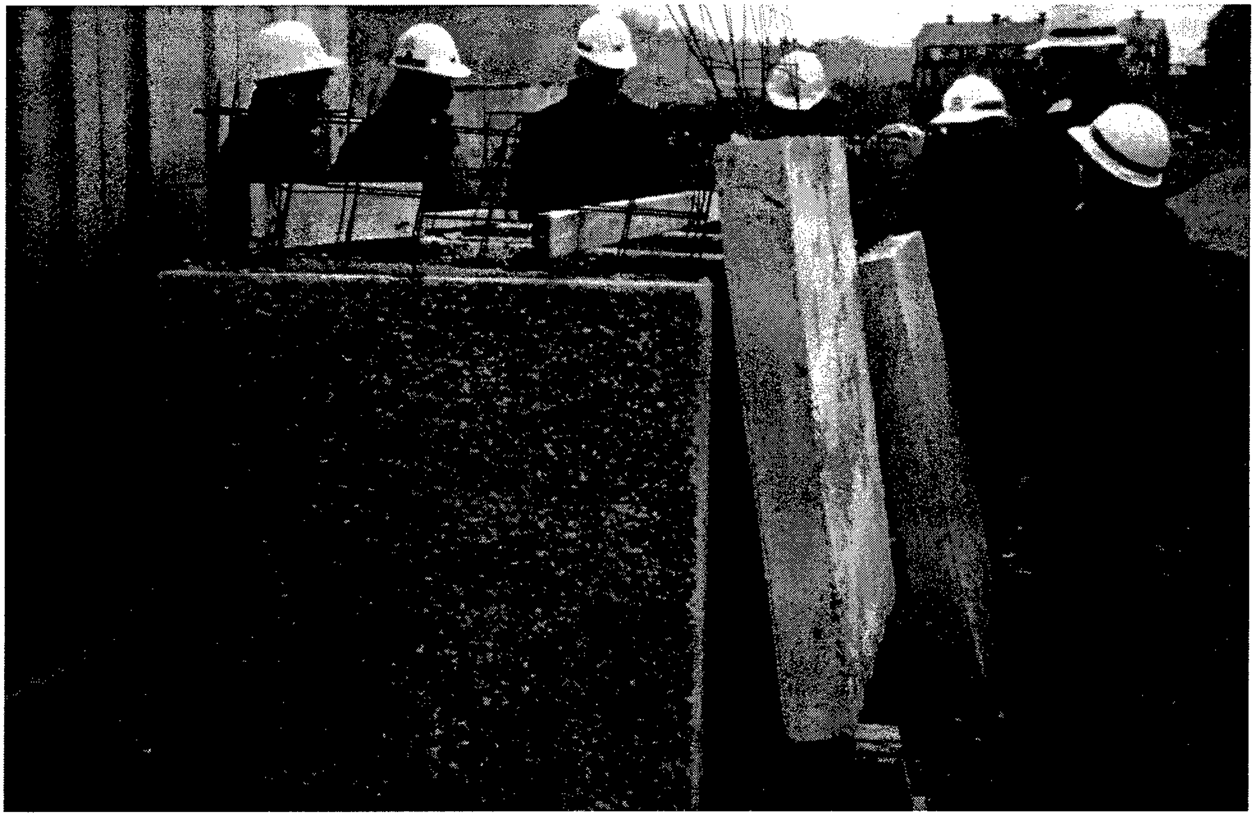


Photo B5. Surface finishes of panels

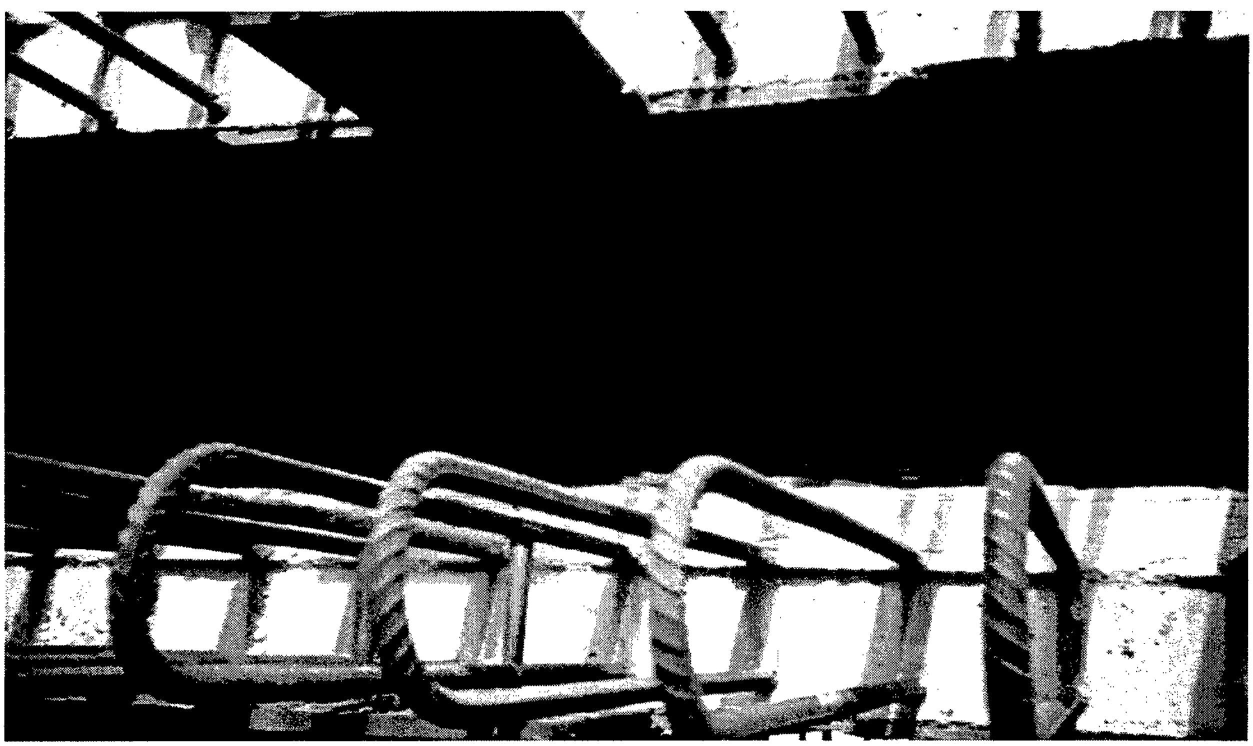


Photo B6. Panel segment reinforcing

Panel Erection

a. Designers should carefully consider panel sizes and develop a maximum panel weight (including reinforcing steel weight) for commercial lifting equipment available on site. Panels over 60 tons greatly reduce the swing radii of crawler cranes, thereby limiting erection efficiency.

b. The design of panel lifting and bracing points should be carefully considered before being left to the contractor to design, as was the case at Braddock. The multiple changes in lifting and bracing plans slowed progress and increased the cost of fabrication, as already installed hardware was later abandoned. Providing the contractor flexibility while ensuring project efficiency, by providing lifting points in the contract design, deserves thought and consideration.

c. Closure pours between panels had very high concentrations of intersecting rebar from the panels, and additional headed reinforcement was added after erection. This resulted in extremely slow rebar installation, difficulty in properly placing and consolidating the concrete, and applying any type of epoxy bonding agent to the panel edges just prior to concrete placement. Most cast-in-place joints were located at the intersection of four panels and hence were critical structural connections. Reinforcement detailing should be considered very closely. Consider the possibility of moving panel joints into areas requiring less reinforcement. An option is to precast the intersections themselves and connect the intersections with cast-in-place joints where only two panels are joining (Photo B7).

d. Detailed consideration of joint design will save time and money by reducing erection costs. Developing connection details that provide stability during erection will reduce or eliminate conflicts with internal embedded items and temporary panel bracing.

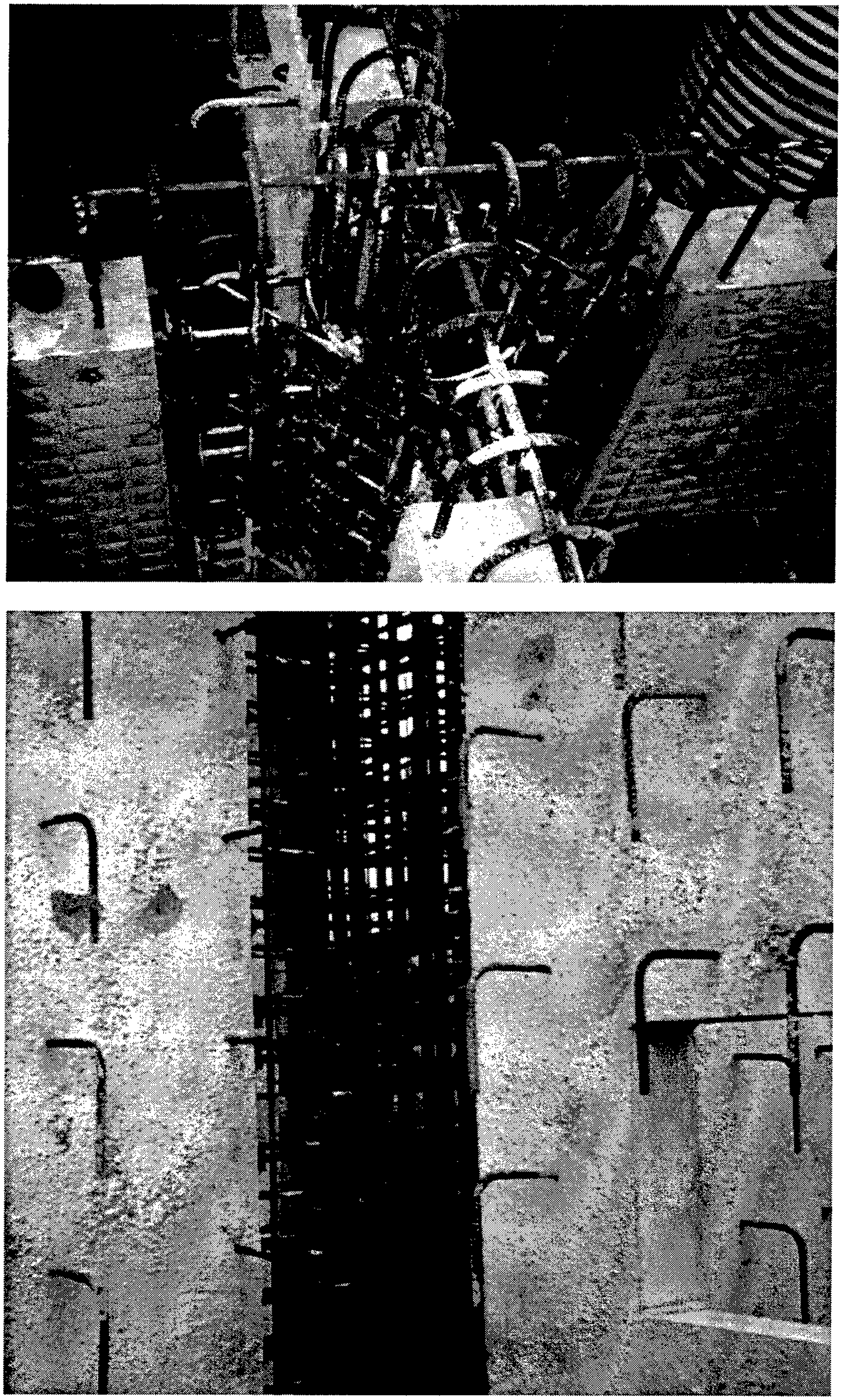


Photo B7. Congested connection of precast panels

Structure Design Considerations

a. Provide adequate access into the structure during construction for safety evacuation in addition to form removal, etc. If adequately planned, the entry access holes can reduce conflicts with other systems such as tremie infill ports or venting holes.

b. Construction tolerances must be realistic for the type of construction undertaken. Clearly indicate the governing tolerances when multiple systems work together. An example is tolerances for location of drilled shaft foundations, the casings within the concrete segments, the seals between the casings and drilled shafts. Another example is thickness tolerances of panels and members with roughened finishes. Clarify the impact of roughened amplitude on overall tolerance, the embedded metal items, and surfacing tolerances. Using continuous reinforcement often encroaches into concrete cover due to manufacturing tolerances and must be considered thoroughly.

c. Require mockups of complicated details and techniques prior to construction. Several applications within float-in structures are different and unique to that project. The mockups used during the Braddock project were very cost-effective for developing work methods able to achieve the quality required. (See Photo B8 for the mockups used for the segment closure pours.)

d. Weight control is a crucial element of success with floating structures. Provide the design weight control model in the contract documents for use during construction, and require continuous updates throughout construction to validate theoretical modeling. Related to this is the weight of concrete used in design estimates. With the variety of complex concrete mixes used on float-in projects, the actual weight of concrete should be developed based on laboratory testing results, and not theoretical or standard values. For the Braddock Dam project, a 1-pcf variance in concrete weight resulted in an overall segment draft change of 1 in.

e. Research various break-away systems for the completed structure during float off. The concrete grade beams, plywood, and plastic sheeting used on the Braddock Dam project represent one example of how the bond between the foundation and the segment may be broken.

f. Post-tensioning duct alignment and type is critical for avoiding conflicts with reinforcement, embedded items, and concrete cover and for reducing cracks during tensioning. Flat ducts require less volume, but tendons must be installed with ducts and potentially remain in place a longer period of time. Designers should clearly identify alignment tolerances and recognize the impact on the construction sequence.

g. Instrumentation details and conduit routings should be done during the initial design to reduce conflicts and achieve desired results. Contractor-

developed designs might favor cost efficiency and might not adequately address data collection requirements or optimize installation sequencing.

h. Openings through top or bottom concrete slabs should be perpendicular to the concrete, which results in a larger opening on sloped surfaces for vertical protrusions. Minimize the number and types of protrusions through slabs, and the overall number of conflicts will be greatly reduced.

i. Thin-walled structures will inherently be heavily reinforced to function as a floating structure. As a result, the effort exerted during the detailing of all reinforcement should take advantage of every possible design tool available. Full-scale drawings and three-dimensional (3-D) detailing are two worthwhile suggestions to reduce what can become continual conflicts during construction. Joints between panels, between panels and top or bottom slabs, and around large embedded items such as drilled shaft foundation casings are the areas of greatest rebar congestion and, at Braddock Dam, resulted in multiple conflicts during construction.

j. Headed rebar designed at closure pours can cause interferences that may have to be remediated by removing some heads or bending some bars to alleviate congestion (Photo B9).

Photo B8. Mockup of segment closure pour

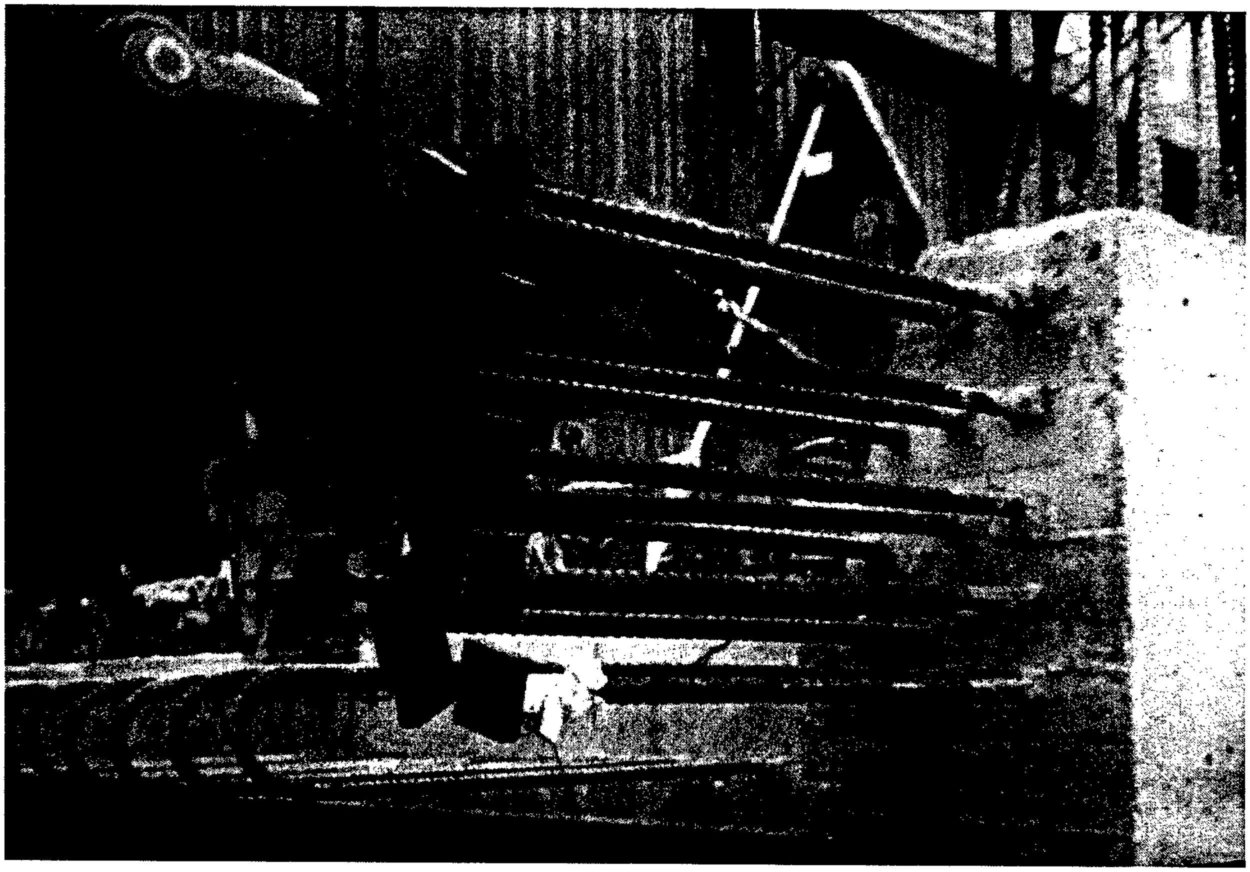


Photo B9. Congestion due to headed reinforcement

Appendix C
Determination of Tolerances

Tolerances for surface leveling, horizontal alignments and vertical alignments of the final position of precast components shall be clearly specified in the plans and specifications. The specified tolerances for the as-built dimensions of the structure should be established on the basis of the requirements for operations, strength, and durability of the structure.

The specified tolerances of the as-built structure are dependent on dimensional tolerances of individual precast products, erection tolerance of the precast components, and interfacial tolerance between adjacent components.

In general, the dimensional tolerances should be used as guidelines for acceptance and not limits for rejection. The engineer shall decide whether a deviation from the allowable tolerance affects safety, service performance, and durability of the structure.

The dimensional tolerance of precast concrete components shall generally comply with the consensus standards of the industry, as described in PCI MNL-116 and PCI MNL-123. For example, dimensional tolerances of a P/C panel include length and width dimension, thickness variation, and straightness (warping, bowing, and tapered edges). Different temperature effects and differential moisture absorption between the inside and outside faces of a panel should be considered in design and construction to minimize warping and bowing.

The erection tolerance shall be determined in accordance with the current construction practice and the project site conditions. The erection tolerance must be economically attainable in construction, especially for underwater positioning of large P/C segments. Tolerances may depend on the site conditions, erection method, and equipment. Final erection tolerances should be reviewed and agreed to prior to initiating erection. Any change in tolerances made from the original plans and specifications shall be approved in writing and noted in the contract documents.

In determination of erection tolerances, attention should be given to possible deflection and/or rotation of the P/C component and the foundation members supporting the P/C component during erection.

The dimensional tolerance and the erection tolerance are interrelated and may be additive. For example, warping, bowing, and edge straightness of a P/C component have an important effect on the edge matchup and joint dimension during the erection. If the accumulated tolerance exceeds the total tolerance specified for the structure, adjustment shall be made with the interface clearance between adjacent precast segments.

Interfacing tolerances and clearances are those required for joining different prefabricated elements. The fabrication dimensional tolerances and erection tolerances must be considered in specifying interfacing tolerances. The clearances between adjacent prefabricated elements are important details, because they provide accommodation for adjusting possible size variation and misalignment.

Typical dimensional tolerances for construction of large precast concrete assemblies are as follows:

Fabrication of individual precast panels:

- Overall dimensions of members: ±1 mm per 1 m of length (1/8 in. per 10 ft), maximum 6 mm (1/4 in.)
- Cross-section dimensions: ±6 mm (1/4 in.)
- Deviation from straight line: ±1 mm per 1 m of length (1/8 in. per 10 ft), maximum 6 mm (1/4 in.)
- Bowing: L/240
- Warpage: one corner out of the plane of the other three shall be less than 5 mm-per-1 m (1/16 in.-per-1 ft) distance from the nearest adjacent corner
- Variation in slab thickness: -13 mm (1/2 in.), +19 mm (3/4 in.)
- Horizontal deviation of ends from square: ±13 mm (1/2 in.)
- Vertical deviation of ends from square: ±13 mm (1/2 in.)

Fabrication of connections:

- Field placed anchor bolts: ±13 mm (1/2 in.)
- Elevation of field cast footing or piers: ±26 mm (1 in.)
- Field placed plates: ±26 mm (1 in.)
- Position of inserts: ±26 mm (1 in.)
- Location of inserts: ±13 mm (1/2 in.)
- Location of bearing plates: ±19 mm (3/4 in.)
- Location of blockouts: ±26 mm (1 in.)
- Bearing deviation from plane: ±5 mm (3/16 in.)

Erection of precast panels to construct large precast concrete assemblies:

- Horizontal tolerance at bottom of erected vertical panels/walls: ±13 mm (1/2 in.)
- Horizontal tolerance at top of erected vertical panels/walls: ±13 mm (1/2 in.)
- Variation from specified bearing length on a support: ±19 mm (3/4 in.)
- Variation from specified bearing width on a support: ±13 mm (1/2 in.)
- Jog in alignment of matching edges: ±13 mm (1/2 in.) maximum

Casting bed:

- Level tolerance of casting bed: ±13 mm (1/2 in.) over entire area

Appendix D
Design Example–Precast Lock Wall Panel

Example calculations have been selected for the design of precast stay-in-place form panels for lock walls at the new Charleroi Locks on the Monongahela River near Pittsburgh. The analysis was performed using the Staad III computer software (not included) and calculations performed in non-SI units. Relevant sections from the PCI Design Handbook (4th ed., 1992) are included within the text.

Methodology

a. Design for the largest panel size and check smaller panels to see if reinforcement is adequate.

Largest panel size: Height = 10'-2"
Width = 8"
Length = 19'-11¾"

b. Material Properties: Concrete: $f'c$ = 5000 psi
γ = 150 pcf

Reinforcement: fy = 60,000 psi

c. Loading Considerations:

(1) *Handling stresses*
- Use 70% $f'c$ = 3500 psi
- Apply static load multiplier of 1.4 (per PCI Table 5.2.1)
  - Lifting stresses (4 pick points)
  - Tilting stresses (2 pick points)

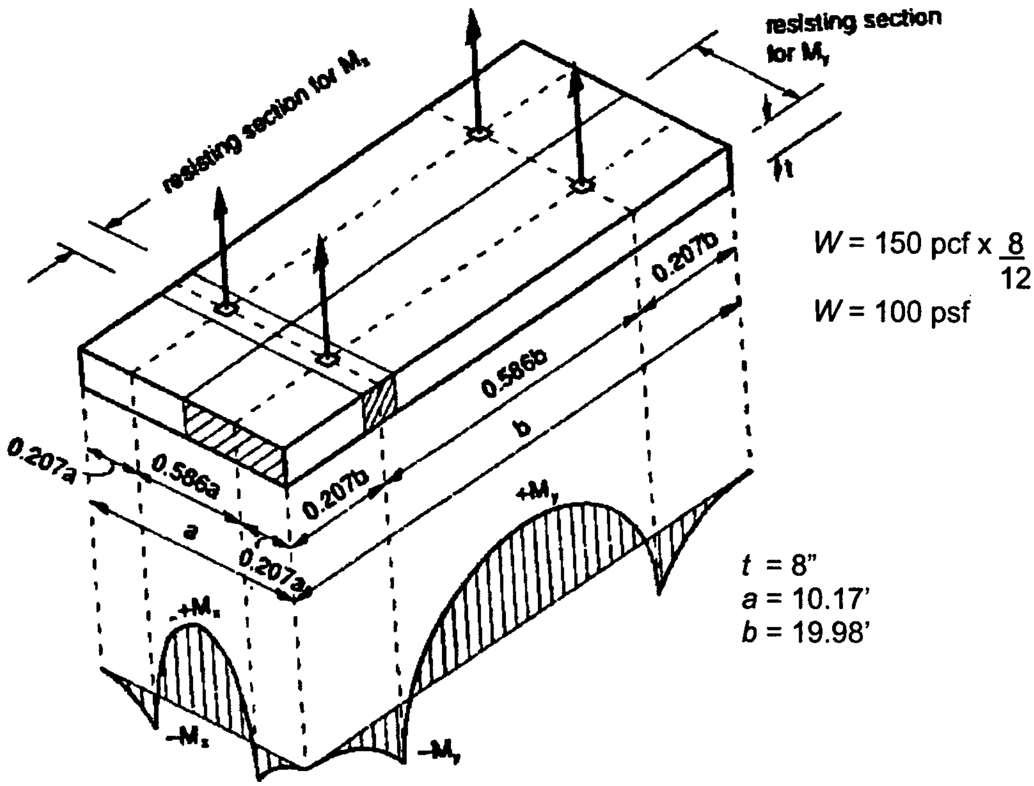
(2) *Infill pressure*
- Use 100% $f'c$ = 5000 psi
- Maximum infill lift = 1/2 panel height
- Apply static load multiplier of 1.4 (per PCI Table 5.2.1)

d. Reinforcement Design:
- Compute minimum reinforcement necessary for each loading considered and design for worst case.

- Use 2-in. cover for surfaces exposed to earth/weather (per EC-1110-2-2104).
- Check crack control based on maximum crack width = 0.006 in.
- Check temperature and shrinkage reinforcement.

(1) Handling stresses

a. Lifting (PCI 5.2)



Maximum Moments: $+M_x = -M_x = 0.0107\ wa^2b$
$M_x = 0.0107$ (100 psf) (10.17')^2 (19.98')
M_x = 2211 lb-ft/ft × 9.99' = 22.088 lb-ft

M_x resisted by smaller of 15 (t) = 15 (8") = 10'
b/2 = 19.98'/2 = 9.99' ← Controls

$+M_y$ = $-M_y = 0.0107\ wab^2$
$M_y = 0.0107$ (100 psf) (10.17') (19.98')^2
M_y = 4344 lb-ft/ft × 10.17'/2
M_y = 22,089 lb. ft (Resisting section for M_Y)

Note: Max. moments could be greater if lifting points not at optimum locations (i.e., 0.207)

Maximum Moments: (compute per 1-ft strip of panel)

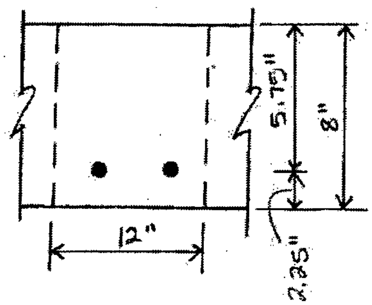
$$M_x = \frac{22.088}{9.99} \times 1\ ft = 2211\ \text{lb-ft/ft}$$

$$M_x = \frac{22.089}{10.17'/2} \times 1\text{ ft} = 4344\text{ lb-ft/ft}$$

Reinforcement:

↓Stripping Loads ↓Load Factor per EM 1110-2-2104 for DL's

$$M_{xu} = 2{,}211\text{ lb.ft/ft} \times 1.4 \times \frac{1}{1000} \times 12 \times 1.7 = 63\text{ k-in}$$



d = 8"-2" – ½ (0.5") (assume No. 4 bar)
d = 5.75"

$$k = 1 - \sqrt{1 - \frac{2\mu/0}{0.85 f'c\, bd^2}} =$$

$$1 - \sqrt{1 - \frac{2(63k - in)/0.9}{0.85(3.5ksi)(12")(5.75)^2}}$$

k = 0.061

$$A_s = \frac{0.85 f'c\, bdk}{f_y} = \frac{0.85(3.5\text{ ksi})(12")(5.75")(0.061)}{60\text{ ksi}} = 0.0210\text{in}^2\text{ft}$$

$$A_{s\,\min} = \frac{200bd}{f_y} = \frac{200(12")(5.75")}{60{,}000} = 0.23\text{in}^2/\text{ft} \quad \text{Controls}$$

$$M_{yu} = 4344\text{ lb-ft/ft} \times 1.4 \times \frac{1}{1000} \times 12 \times 1.7 = 124\text{ k-in}$$

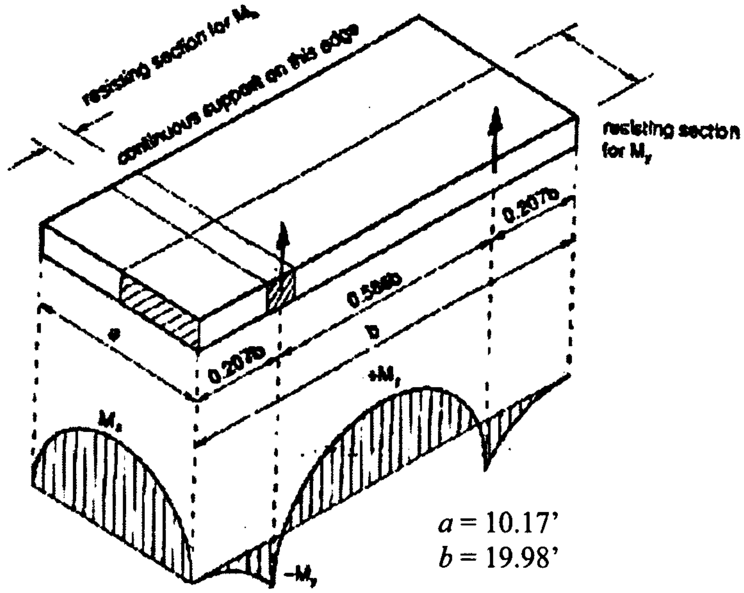
d = 5.75"

$$k = 1 - \sqrt{1 - \frac{2(124\text{k-in})/0.9}{0.85(12)(5.75)^2(3.5)}} = 0.124$$

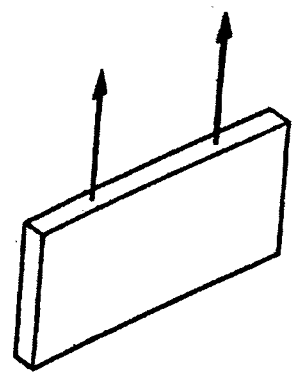
$$A_s = \frac{0.85(3.5)(12")(5.75)(0.125)}{60} = 0.42\text{in}^2/\text{ft} \quad \text{Controls}$$

$$A_{s\,\min} = 0.23\text{in}^2/\text{ft}$$

b. Tilting (PCI 5.2)



$a = 10.17'$
$b = 19.98'$



$$W = 150\text{ pcf} \times \frac{8"}{12} = 100\text{ psf}$$

Maximum Moments: $M_x = \dfrac{wa^2}{8} = \dfrac{(100\text{ psf})(10.17')^2}{8}$

$M_x = 1{,}293$ lb-ft/ft
$M_y = 0.0107\ wab^2 = 0.0107\ (100\text{ psf})\ (10.17')\ (19.98')^2$
$M_y = 4344$ lb-ft/ft $\times$ 10.17'/2
$M_y = 22{,}090$ lb-ft (Resisting section for M_y

Maximum Moments: (Per 1-ft strip of panel)

$$1{,}293\text{ lb-ft/ft} \times 1.4 \times \frac{1}{1000} \times 12 \times 1.7 = 37\text{ k-in}$$

(1,293 = Stripping Loads) (1.4 = Load Factor for DL)

$$M_{yu} = \frac{22{,}090\text{ lb-ft}}{10.17"/2} \times 1' \times 1.4 \times \frac{1}{1000} \times 12 \times 1.7 = 124\text{ k-in}$$

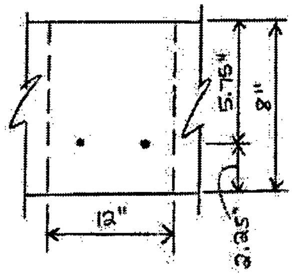
Reinforcement:

$M_{xu} = 37$ k-in

$d = 8" - 2" - \frac{1}{2}(0.5") = 5.75"$ *(Assume #4 Bar)*

$$k = 1 - \sqrt{1 - \frac{2(37\text{ k-in})/0.9}{0.85(3.5)(12")(5.75")^2}}$$

$k = 0.035$



$$A_s \max = \frac{0.85(3.5\text{ ksi})(12")(5.75")(0.035)}{60\text{ ksi}} = 0.120\text{ in}^2/\text{ft}$$

$$A_s \min = \frac{200(5.75")(12")}{60{,}000} = 0.23\text{ in}^2/\text{ft} \quad \text{Controls } (M_x)$$

$M_{yu} = 124$ k.in

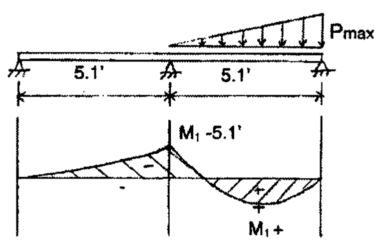
$d = 5.75"$

$$k = 1 - \sqrt{1 - \frac{2(124)/0.9}{0.85(3.5)(12")(5.75")^2}} = 0.124$$

$$A_s \max = \frac{0.85(3.5\text{ ksi})(12")(5.75")(0.124)}{60\text{ ksi}} = 0.42\text{ in}^2/\text{ft} \qquad \text{Controls } (M_y)$$

$A_s \min = 0.23\text{ in}^2/\text{ft}$

(2) **Infill Stresses**
Panel height = 10.17'
Pour heights on panels limited to ½ panel height = 10.17'/2 = 5.1'
Treat panel as a simple beam with a double span



POUR 1

$Pmax = 0.150\text{ kcf} \times 5.1' = 0.765\text{ ksf}$

Approximate Moments:

$0.765\text{ ksf} \times 2/3 = 0.51$

$$M_{1^-} = \frac{wl^2}{16} = \frac{(0.51)(5.1')^2}{16} = 0.83\text{ k-ft}$$

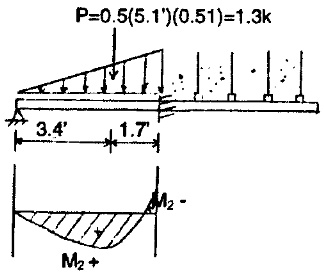
$$M_{1^+} = \frac{49}{512} wl^2 = \frac{49}{512}(0.51)(5.1)^2 = 1.27\text{ k-ft}$$

$$M_{2^-} = \frac{pab}{21^2}(a+1) = \frac{(1.3)(3.4')(1.7')(3.4'+5.1)}{2(5.1)^2}$$

$M_2- = 1.23$ k-ft

$$M_{2^+} = \frac{pb^2}{21^3} - (a+21)(a) = \frac{(1.3)(1.7')^2(3.4'+2(5.1))}{2(5.1)^3 \times \frac{1}{3.4'}}$$

$M_2+ = 0.65$ k-ft



Add stresses from Pours 1 and 2:

$M+$ total = 0.83 k-ft + 0.65 k-ft = 1.92 k-ft

$M-$ total = 0.83 k-ft + 1.23 k-ft = 2.06 k-ft ← Controls

Verify moments obtained by approximation using equations from AISC (9th ed.) with a STAAD Model (not included in appendix) of the precast panel with infill stresses (Pour 1 only).

Per STAAD output, $Mmax$ = 1.05 k-ft < $Mmax$ = 1.27 k-ft (Pour 1) – Use moments obtained through approximate method.

$M\text{-}total_u = 2.06$ k-ft/ft × 1.7 × 12 = 42 k-in
(1.7 = Load Factor for DL per EM 1110-2-2104)

Reinforcement:

$Mmax_u = 42$ k-in

$d = 8" - 2" - ½ (0.5") = 5.75"$

$$k = 1 - \sqrt{1 - \frac{2(42\text{ k-in})/0.9}{0.85(5\text{ ksi})(12")(5.75")^2}} = 0.028$$

$$A_s = \frac{0.85(5\text{ ksi})(12")(5.75")(0.028)}{60\text{ ksi}} = 0.14\text{ in}^2/\text{ft}$$

$$A_s\min = \frac{200(12")(5.75")}{60{,}000} = 0.23\text{ in}^2/\text{ft}$$ Controls

Temperature and Shrinkage Reinforcement:

Per EM 1110-2-2104:

$A_s = 0.0028 \times A_g$ (half in each face)

$A_g = 8" \times 12" = 96\text{ in}^2$

$A_s = 0.0028 \times 96\text{ in}^2 = 0.270\text{ in}^2/\text{ft}$

$A_s/2 = 0.270/2 = 0.140\text{ in}^2/\text{ft}$ in each face

Summary:

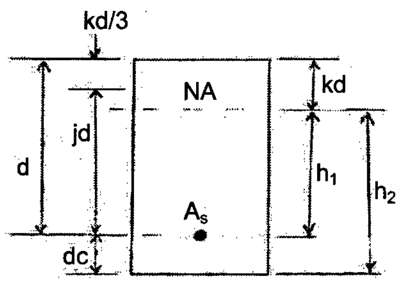
	Reinforcement Needed
Handling stresses Lifting Tilting	 0.42 in2/ft 0.42 in2/ft
Infill stresses	0.23 in2/ft (M-controlled)
Temperature and shrinkage	0.14 in2/ft

Try No. 4 bars at 6-in. spacing → A_s prov. = 2(0.20 in^2/ft).

- Check crack control using maximum service load moments.

Crack Control

Max. Service Load Moments = 124 k-in/1.7 = 73 k-in



d = 8" – 2" cover – ½ (0.5") = 5.75"

$A_s = \underline{(0.20\text{ in}^2)(12)} = 0.40\text{ in}^2/\text{ft}$

$$p = \frac{As}{bd} = \frac{0.40\text{ in}^2/\text{ft}}{(12")(5.75")} = 0.0058$$

$$n = \frac{\text{Esteel}}{\text{Econc}} = \frac{29{,}000\text{ ksi}}{4{,}100\text{ ksi}} = 7$$

(f'c = 5000 psi (28-day compressive strength))

$np = (7)(0.0058) = 0.0406$

$$k = \sqrt{(np)^2 + 2np} - np = \sqrt{(0.0406)^2 + 2(0.0406)} - (0.0406)$$

$k = 0.247$

$kd = (0.247)(5.75") = 1.42$ $\qquad kd/3 = 0.473$

$j = 1 - k/3 = 1 - 0.247/3 = 0.918$

$jd = (0.918)(5.75") = 5.28"$

$$fc = \frac{2M}{kjd^2b} = \frac{2(73\text{ k-in})}{(0.247)(0.918)(5.75)^2(12)} = 1.62\text{ ksi} < .45(7000\text{ ksi}) = 3.2\text{ ksi} \qquad \textbf{OK}$$

$$fs = \frac{M}{As\ jd} = \frac{(73\text{ k-in})}{(0.40)(5.28)} = 34.6\text{ ksi} > 24\text{ ksi} \qquad \rightarrow \textbf{NO GOOD}$$

(Try #5 Bars @ 6" Spacing)

- Check crack control for No. 5 bars at 6-in. spacing.

$d = 8" - 2" - ½ (0.625) = 5.69"$

$$A_s = \frac{(0.13\text{ in}^2)(12)}{6} = 0.62\text{ in}^2/\text{ft}$$

$$p = \frac{0.62\text{ in}^2/\text{ft}}{(12")(5.69")} = 0.0091$$

$n = 7$

$np = (7)(0.0091) = 0.064$

$$k = \sqrt{(0.064)^2 + 2(0.064)} - (0.064) = 0.299$$

$kd = (0.299)(5.69) = 1.70 \qquad kd/3 = 0.567$

$j = 1 - 0.299/3 = 0.900$

$jd = (0.900)(5.69) = 5.12$

$$f_c = \frac{73\text{ k-in} \times 2}{(0.299)(0.900)(5.69)^2(12)} = 1.40\text{ ksi} < 3.2\text{ ksi}$$ **OK**

$$f_s = \frac{73\text{ k-in}}{(0.62)(5.12)} = 23\text{ ksi} < 24\text{ ksi}$$ **OK**

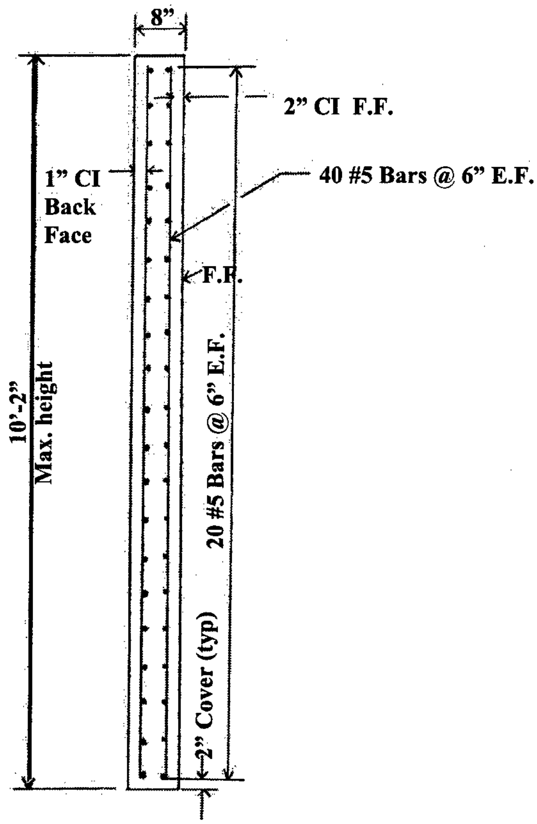
$$a = \frac{2(12)(2.31)}{2} = 28\text{ in}^2 \qquad h_2 = 8 - 1.60 = 6.4" \qquad h_1 = 8 - 1.60 - 2.31 = 4.09"$$

$w = 0.000076\ (6.4"/4.09")\ (11.0)\ (2.31 \times 29)^1/3$

$= 0.0053" < 0.006"$ **OK**

USE #5 BARS @ 6" SPACING EACH WAY, EACH FACE

TYPICAL SECTION Largest Panel Size – 10.71' × 19.98' × 8"



References

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PCI Design Handbook, 4th ed., 1992

EM 1110-2-2104, "Strength Design for Reinforced Concrete Hydraulic Structures," June 1992

EC 1110-2-6052, "Structural Design of Precast and Prestressed Concrete for Offsite Prefabricated Construction of Hydraulic Structures," Jan 2001

Appendix E
Design Examples

Design Example 1

A gated navigation dam structure (183 m (600 ft) long) with four tainter gate bays will be constructed with the float-in method. The "In-the-Wet" construction plan calls for breaking the dam into two segments—102 m (333 ft) long and 81 m (265 ft) long. The segments will be constructed as closed-bottom precast concrete box structures. The bottom of the boxes will be recessed to fit the preinstalled foundation caissons on site. As each segment module is fabricated in a casting yard, it will be launched by flooding and towed to the site for final outfitting. It will then be positioned over the foundation caissons with a mooring system mounted on the segment. Each module will be ballasted down onto six landing caissons and leveled with flat jacks. The pile tops and underbase will be grouted, and 3.4-m (11-ft)-thick tremie concrete will be placed in the segment module. Each module will then be dewatered, and the remainder of the dam including tainter gates will be completed in the dry.

This example illustrates the design procedure to check the flexural, shear, and cracking of the reinforced and prestressed concrete sections in the float-in dam segment. The load case occurs during the transport stage of precast concrete dam segments from a casting yard to the outfitting site.

These calculations are performed using AASHTO LRFD Bridge Specifications, SI units (2nd ed., 1998). ***All equation numbers refer to this guide.*** All SI units are in newtons (N) and millimeters (mm) unless otherwise noted.

The design cases are as follows:

- Flexural check of a reinforced concrete section.
- Shear check of a reinforced concrete section.
- Flexural and cracking check of a prestressed concrete section.

Demand calculations

First, the demand moments and shears must be found over the segment. A finite element model of the segment is created to find the appropriate values.

The finite element analysis is performed by using the SAP2000 nonlinear version 6.11 program from Computers and Structures, Inc., Berkeley. The nonlinear features in the program are not used during the analysis.

Grid models constructed by beam elements simulate the behavior of the dam. The properties of the elements are changed as the dam structure changes for different stages. The center line of the frame elements is located at the center line of the webs.

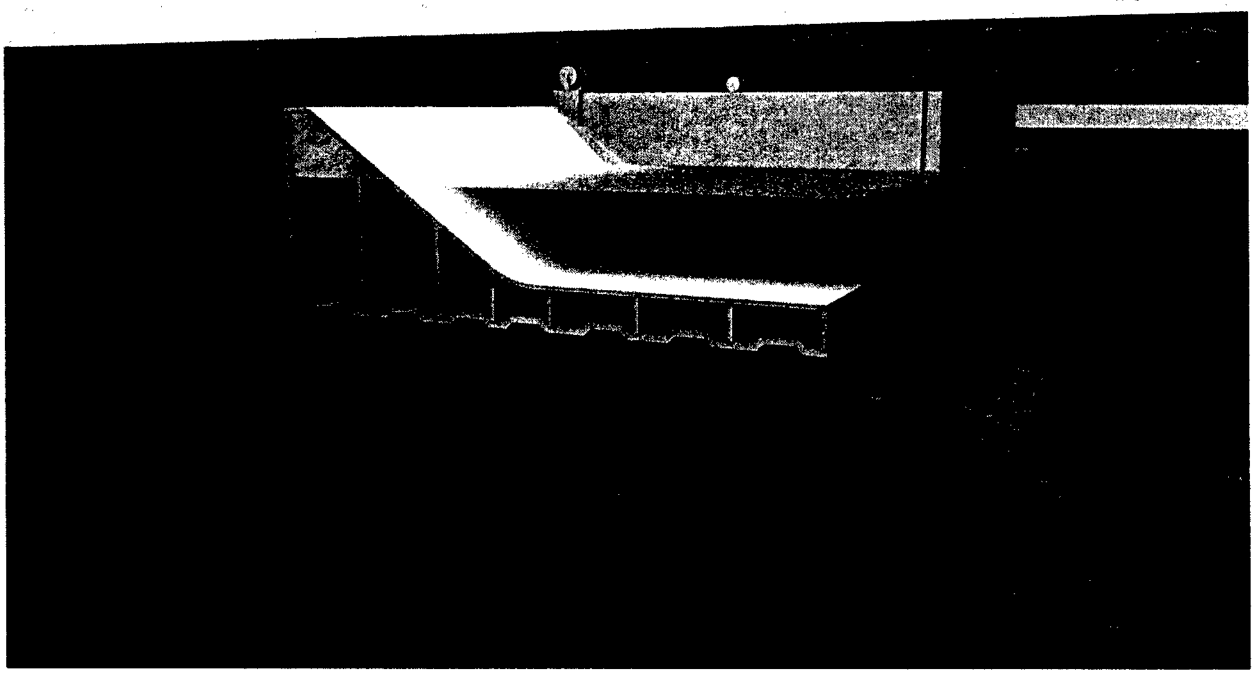


Figure E.1. Diagram of dam being lowered onto foundation

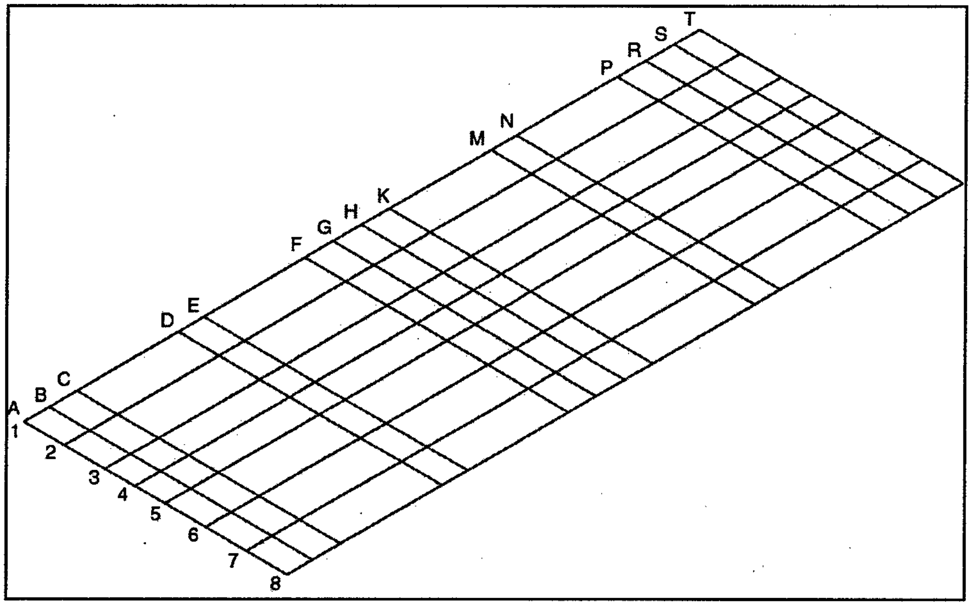


Figure E.2. Labels of equivalent frame members

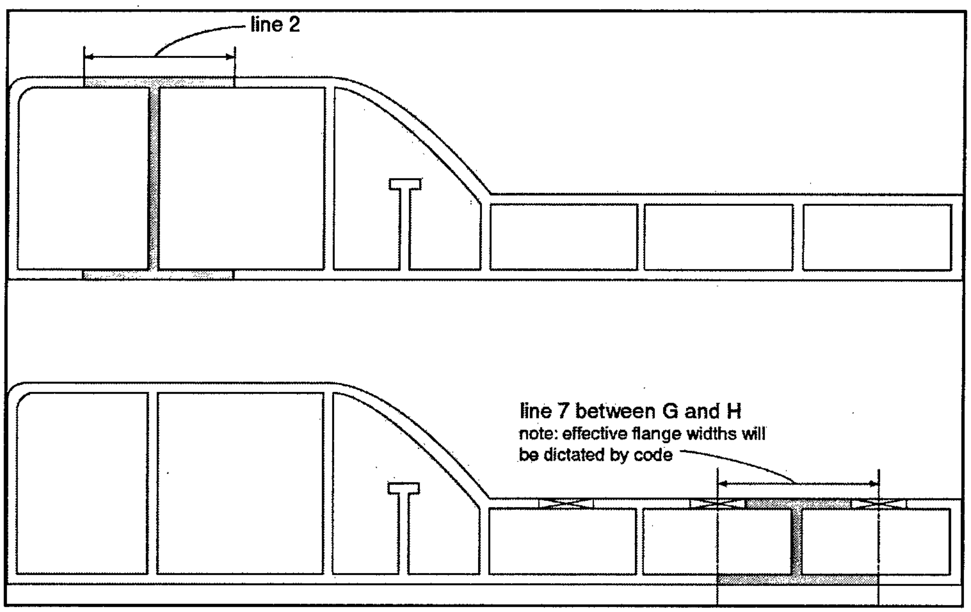


Figure E.3. Diagram of frame sections used during analysis

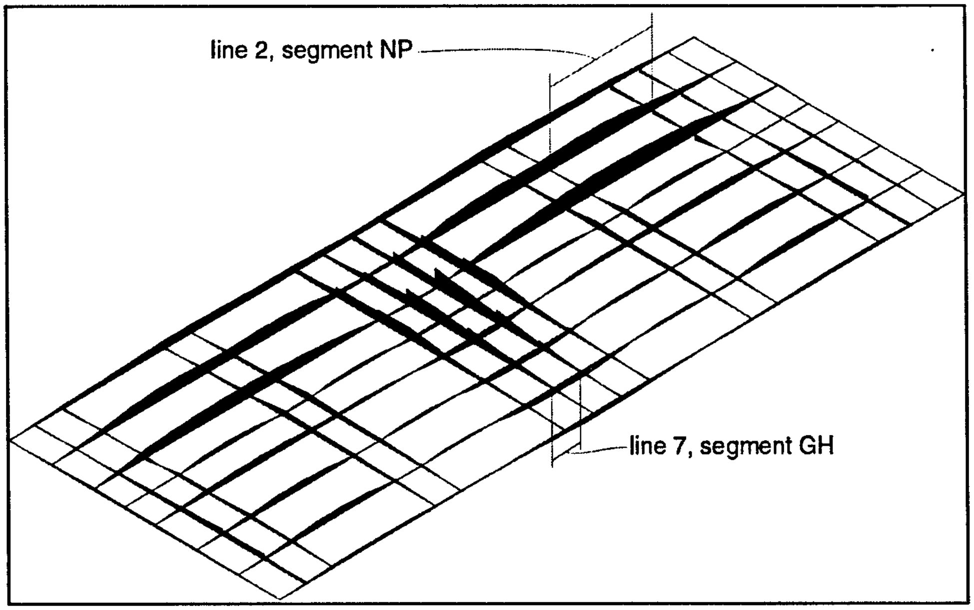


Figure E.4. Bending moments on frame model during float-out load case

The buoyancy while the dam is floating is modeled by springs located at each node location. The spring stiffnesses are equivalent to the tributary area multiplied by the density of the water, thereby simulating the buoyancy.

The demand for the three cases:

- Flexural strength check of critical section -
  Line 2, segment NP – M_u = 3,640 kN-m

- Shear strength check of critical section -
  Line 2, segment NP - V_u = 863 kN

- Flexural and cracking strength check of prestressed concrete section –
  Line 7, segment GH - M_u = -2,422 kN-m

A load factor of 1.7 has been used for all moments and shears as it is a hydrostatic load case.

Capacity calculations

Flexural check of reinforced concrete section

Use line 2, segment NP (Figure E.4). The flexural strength of this segment must be checked against the demand moment calculated during the float-out process (LC 1). The capacity will be calculated according to AASHTO section 5.7.3 – Flexural Members. The segment is simplified as a wide-flange beam with the following assumptions:

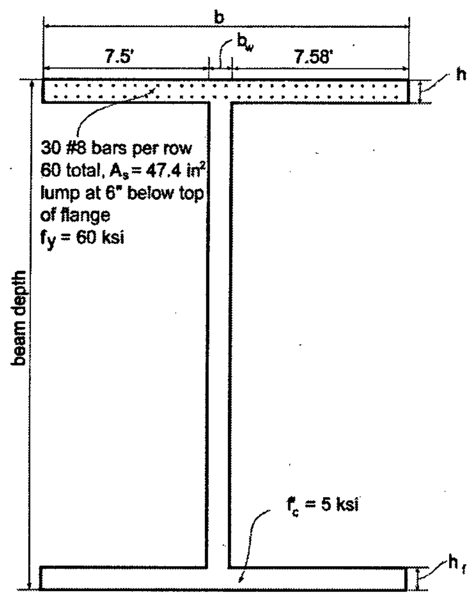
The tension steel consists of 60 No. 8 bars. The steel is lumped at its centroid, 6 in. from the edge of the flange. The steel in the web and the compression steel are ignored, a conservative estimate.

Calculate effective flange width. Taken as the least of three values (4.6.2.6.1):

- one-quarter of the effective span length = ¼ × 135 ft = 33.75 ft
- 12.0 times the average thickness of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder = 12 × 1 ft + ½ × 7.5 ft = 16 ft
- the average spacing of adjacent beams = 16 ft

Use 16 ft.

The beam has the following properties:



Property	*Non-SI Units*	*SI Units*
A_s	47.4 in^2	30,581 mm^2
f_y	60 ksi	414 MPa
Beam depth	21 ft	6,401 mm
f_c	5 ksi	34 MPa
B	15.92 ft	4,852 mm
b_w	0.83 ft	253 mm
h_f	1 ft	305 mm
Cover	0.5 ft	152 mm
d_s	20.5 ft	6,248 mm

Calculate β_1, stress block factor:

$$\beta_1 = 0.85 - \frac{0.05 \cdot (f'_c - 28)}{7} = 0.85 - \frac{0.05 \cdot (34 - 28)}{7} = 0.804 \qquad (5.7.2.2)$$

Assuming the section behaves as a rectangular beam, calculate c, distance between the neutral axis and the compressive face:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{30{,}581 \cdot 414}{0.85 \cdot 34 \cdot 0.804 \cdot 4852} = 111\, mm < h_f \qquad (5.7.3.1.1\text{-}4)$$

The assumption of behavior as a rectangular beam rather than a T-beam is correct because c is less than h_f, and the compression zone does not extend into the web.

Calculate a, depth of equivalent stress block:

$$a = c \cdot \beta_1 = 111 \cdot 0.804 = 89\ mm \qquad (5.7.2.2)$$

Find nominal flexural resistance:

$$M_n = A_s \cdot f_y \cdot (d_s - \frac{a}{2}) = 30{,}581 \cdot 414 \cdot (6248 - \frac{89}{2}) = 7.848 \cdot 10^{10}\ N{-}mm \qquad (5.7.3.2.2\text{-}1)$$

Find factored flexural resistance:

$\varphi = 0.90$ for flexure and tension of reinforced concrete beams (5.5.4.2.1)

$$M_r = \varphi \cdot M_n = 0.9 \cdot 7.848 \cdot 10^{10} = 7.06 \cdot 10^{10}\ N{-}mm \qquad (5.7.3.2.1\text{-}1)$$

Check demand versus capacity:

$$M_r > M_u\ ?$$
$$70{,}600\ kN{-}m > 3{,}640\ kN{-}m$$

OK

2) Check of shear in line 2, segment NP:

The cross section is the same as Example 1. Shear reinforcement consists of No. 7 stirrups spaced at 9 in.

Calculate A_v, steel area:

$$A_v = 0.60\ in^2 \cdot 2 = 1.20\ in^2$$

Find b_v, effective web width, and d_v, effective shear depth:

$$b_v = web\ thickness = 0.83\ ft$$

$$d_v = d_s - \frac{a}{2} = 20.4\ ft$$

The resistance factor for shear, φ, is 0.9 for normal-density concrete in accordance with AASHTO LRFD Specifications (5.5.4.2.2).

Property	Non-SI Units	SI Units
A_v	1.2 in^2	774 mm^2
S	0.75 ft	229 mm
E_s	29,000 ksi	2×10^5 MPa
b_v	0.83 ft	253 mm
d_v	20.4 ft	6,204 mm
φ	0.9	0.9

To find V_n, nominal shear resistance, take the lesser of

1) $V_n = 0.25 \cdot f_c^{'} \cdot b_v \cdot d_v$ (5.8.3.3-1)

2) $V_n = V_c + V_s + V_p$ (5.8.3.3-2)

Calculate first term:

$$V_n = 0.25 \cdot f_c^{'} \cdot b_v \cdot d_v = 0.25 \cdot 34 \cdot 253 \cdot 6204 = 13{,}527\ kN$$

Calculate second term:

V_p, the vertical component of the prestressing force = 0

To calculate V_s and V_c, assume a value for θ, angle of inclination of transverse reinforcement to longitudinal axis, and β, factor indicating ability of diagonally cracked concrete to transmit tension:

θ	32
β	3.5

Calculate V_c and V_s:

$$V_c = 0.083 \cdot \beta \cdot \sqrt{f_c^{'}} \cdot b_v \cdot d_v = 0.083 \cdot 3.5 \cdot \sqrt{34} \cdot 253 \cdot 6204 = 2677\ kN \quad (5.8.3.3\text{-}3)$$

$$V_s = \frac{A_v \cdot f_y \cdot d_v \cdot \cot\theta}{s} = \frac{774 \cdot 414 \cdot 6204 \cdot \cot(32)}{229} = 13{,}909\ kN \quad (C5.8.3.3\text{-}1)$$

Find v, the shear stress on the concrete:

$$v = \frac{V_u - \varphi \cdot V_p}{\varphi \cdot b_v \cdot d_v} = \frac{862{,}746 - 0.9 \cdot 0}{0.9 \cdot 253 \cdot 6204} = 0.611\ MPa \qquad (5.8.3.4.2\text{-}1)$$

and

$$\frac{v}{f_c^{'}} = \frac{0.611}{34} = 0.018$$

Calculate ε_x, the strain in the reinforcement on the flexural tension side:

$$\varepsilon_x = \frac{\frac{M_u}{d_v} + 0.5 \cdot V_u \cdot \cot\theta}{E_s \cdot A_s} \qquad (5.8.3.4.2\text{-}2)$$

$$= \frac{\frac{3{,}640{,}000}{6204} + 0.5 \cdot 862{,}746 \cdot \cot(32)}{2x10^5 \cdot 30{,}581} = 2.1 \cdot 10^{-4}$$

Checking the values on the charts: (see following page)

If $v/f_c^{'} = 0.018$ and $\varepsilon_x = 2.1 \cdot 10^{-4}$, then $\beta = 3.6$ and $\theta = 28$ deg. Need to guess new values for β and θ.

Try: $\beta = 3.6$ and $\theta = 28$ deg

Thus,

$$V_c = 2{,}677kN$$
$$V_s = 16{,}347kN$$
$$v = 0.611MPa$$
$$\varepsilon_x = 2.3x10^{-4}$$

If $v/f_c^{'} = 0.018$ and $\varepsilon_x = 2.3 \cdot 10^{-4}$, then $\beta = 3.6$ and $\theta = 28$ deg, so estimates are good.

$$V_n = V_c + V_s + V_p = 2{,}677kN + 16{,}347kN + 0kN = 19{,}024\,kN$$
$$19{,}024\,kN > 13{,}527kN$$
$$Use\ 13{,}527\,kN$$

$$V_r = \varphi \cdot V_n = 0.9 \cdot 13{,}527kN = 12{,}174kN$$

Check demand versus capacity:

$V_r > V_u$?

$12{,}174\,kN > 863\,kN \qquad OK$

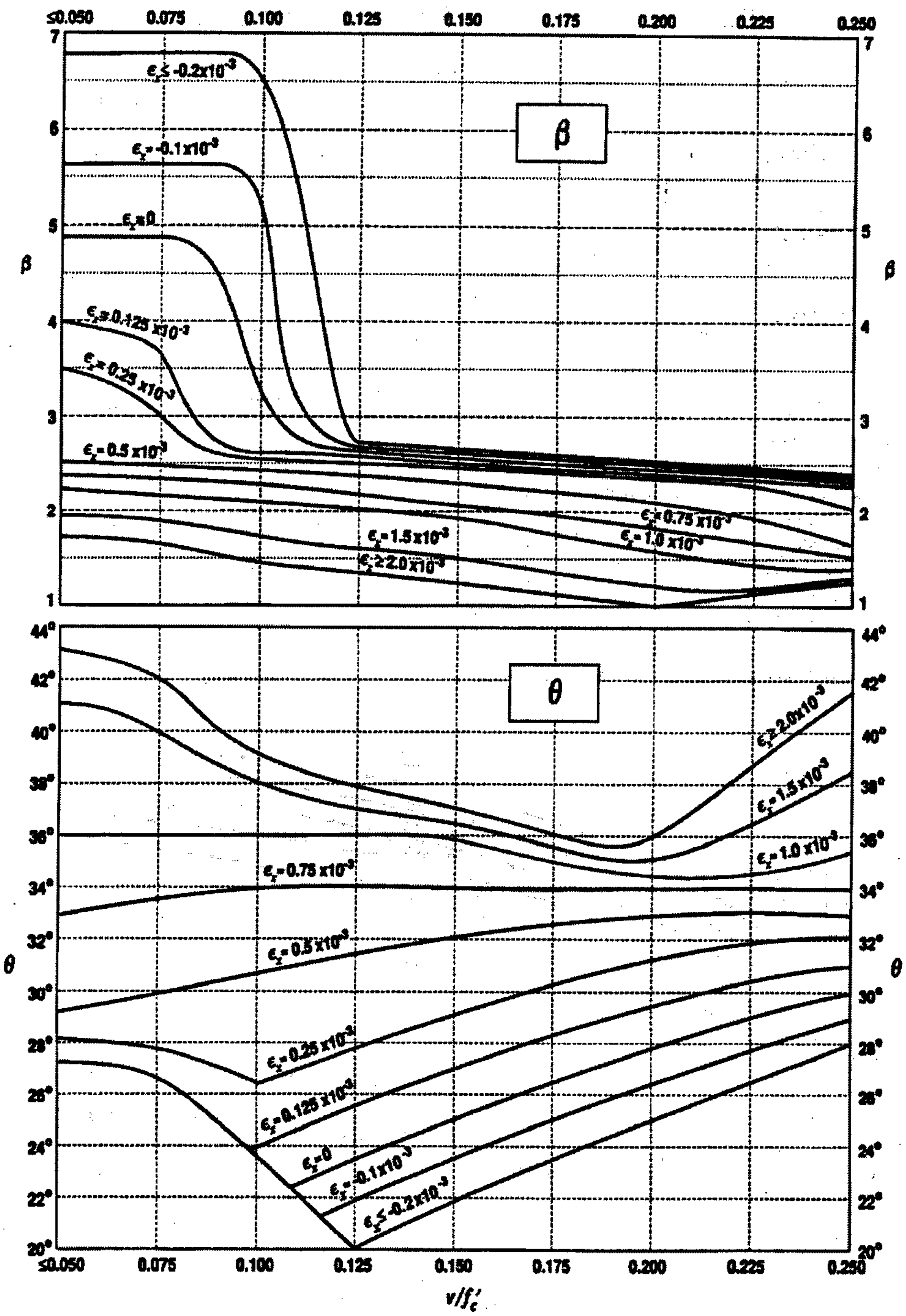


Table 5.8.3.4.2-1. Values of θ and β for sections with Transverse Reinforcement

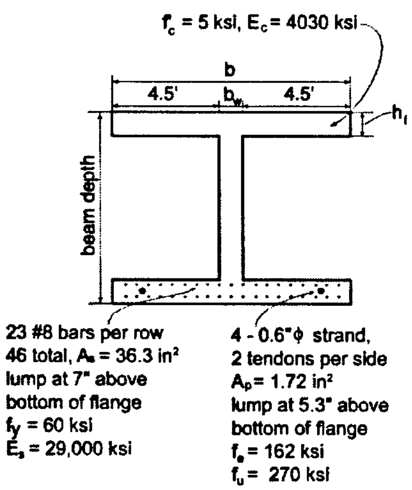
3) Check bending moment and crack width in prestressed beam - line 7, segment GH:

Calculate effective flange width. Taken as the least of three values (4.6.2.6.1):

- one-quarter of the effective span length = ¼ × 40 ft = 10 ft
- 12.0 times the average thickness of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder = 12 ft + ½ × 5.75 ft = 15 ft
- the average spacing of adjacent beams = 16 ft

Use 10 ft.

The prestressed beam has the following properties:



Property	***Non-SI Units***	***SI Units***
A_s	36.3 in^2	23,445 mm^2
f_y	60 ksi	414 MPa
E_s	29,000 ksi	2×10^5 MPa
Beam depth	8 ft	2,438 mm
f'_c	5 ksi	34 MPa
b	10 ft	3,048 mm
b_w	1.0 ft	305 mm
h_f	1.0 ft	305 mm
Cover	7 in	178 mm
d_s	7.42 ft	2,260 mm
A_{ps}	1.72 in^2	1,110 mm^2
f_{pe}	162 ksi	1,117 MPa
d_p	7.56 ft	2,303 mm

Calculate β_1, stress block factor:

$$\beta_1 = 0.85 - \frac{0.05 \cdot (f'_c - 28)}{7} = 0.85 - \frac{0.05 \cdot (34 - 28)}{7} = 0.804 \qquad (5.7.2.2)$$

Find f_{py}, the yield strength of the prestressing tendons:

$$f_{py} = 0.90 \cdot f_{pu} = 0.90 \cdot 1860 = 1675 MPa \qquad (C5.7.3.1.1\text{-}1)$$

Find k:

$$k = 2\cdot\left(1.04-\frac{f_{py}}{f_{pu}}\right) = 2\cdot\left(1.04-\frac{1675}{1862}\right) = 0.28 \qquad (5.7.3.1.1\text{-}2)$$

Assuming the section behaves as a rectangular beam, calculate c, the distance between the neutral axis and the compressive face:

$$c = \frac{A_{ps}\cdot f_{pu} + A_s\cdot f_y}{0.85\cdot f_c'\cdot\beta_1\cdot b + k\cdot A_{ps}\cdot\dfrac{f_{pu}}{d_p}} \qquad (5.7.3.1.1\text{-}4)$$

$$= \frac{1110\cdot 1861 + 23{,}455\cdot 414}{0.85\cdot 34\cdot 0.804\cdot 3048 + 0.28\cdot 1110\cdot\dfrac{1860}{2303}} = 163\ mm$$

Find f_{ps}, the average stress in prestressing steel:

$$f_{ps} = f_{pu}\cdot\left(1-k\cdot\frac{c}{d_p}\right) = 1861\cdot\left(1-0.28\cdot\frac{63.3}{2303}\right) = 1824 MPa \qquad (5.7.3.1.1\text{-}1)$$

Calculate a, depth of equivalent stress block:

$$a = c\cdot\beta_1 = 163\cdot 0.804 = 131\ mm \qquad (5.7.2.2)$$

Find nominal flexural resistance:

$$M_n = A_s\cdot f_y\cdot\left(d_s-\frac{a}{2}\right) + A_{ps}\cdot f_{ps}\left(d_p-\frac{a}{2}\right)$$

$$= 23{,}455\,(414)\cdot\left(2260-\frac{131}{2}\right) + 1110\cdot 1824\cdot\left(2303-\frac{131}{2}\right) \qquad (5.7.3.2.2\text{-}1)$$

$$= 2.58\cdot 10^{10}\ N-mm$$

Find factored flexural resistance:

$\varphi = 1.0$ for flexure and tension of prestressed concrete

$$M_r = \varphi\cdot M_n = 1.0\cdot 2.58\cdot 10^{10} = 25{,}800\ kN-m \qquad (5.7.3.2.1\text{-}1)$$

Check demand versus capacity:

$M_r > M_u$?

25,800 $kN-m$ >2422 $kN-m$ OK

The moment capacity is sufficient, but must also check that there is no cracking or small cracking in the base of the slab due to the bending moment.

To calculate the stress in the bottom flange of the beam:

$$\sigma_{total} = \sigma_d + \sigma_{pa} + \sigma_{pb}$$

The tensile stress due to the demand bending moment is

$$\sigma_d = \frac{M_d \cdot y}{I} = \frac{2.42 \cdot 10^9 \cdot 1219}{2.29 \cdot 10^{12}} = 0.93MPa$$

where

y = distance from the centroid to the extreme fiber = 1,219 mm
I = moment of inertia of the section

The compressive stress due to the axial force applied by the prestressing tendons is

$$\sigma_{pa} = \frac{-P}{A} = \frac{1.24 \cdot 10^6}{2.42 \cdot 10^6} = -0.51MPa$$

where

P = axial force applied by the prestressing tendons = 1,240 kN
A = area of the section = 2.42×10^6 mm^2

The compressive stress due to the bending moment applied by the prestressing force is

$$\sigma_{pb} = \frac{-P \cdot e \cdot y}{I} = \frac{1.24 \cdot 10^6 \cdot 1084.26 \cdot 1219}{2.29 \cdot 10^{12}} = -0.51MPa$$

where

e = distance from the prestressing tendons to the centroid of the section
σ_{total} = 0.93 – 0.51 – 0.51 = -0.09 MPa (compression)

Thus, because there is no tension in the bottom of the slab, there is no cracking. If there were tension, it would have to be checked according to provision 5.7.3.4.

Design Example 2

This example is to perform a calibration check of Design Example 1 with the ACI design provisions. This example addresses the same design problems as those in Example 1, but using ACI 318M-99, "Building Code Requirements for Structural Concrete and Commentary." All the equation numbers refer to those in ACI 318-99. All SI units are in N and mm unless otherwise noted.

Demand calculations

A load factor of 1.7 has been used for all moments and shears.

- Flexural strength check of critical section - Line 2, segment NP
  M_u = 3,640 kN-m

- Shear strength check of critical section - Line 2, segment NP
  V_u = 863 kN

- Flexural and cracking strength check of prestressed concrete section – Line 7, segment GH
  M_u = -2,422 kN-m

Capacity calculations

1) Flexural check of reinforced concrete section:

Use line 2, segment NP. The flexural strength of this segment must be checked against the demand moment calculated during the float-out process (LC1). The segment is simplified as a wide-flange beam with the following assumptions:

The tension steel consists of 60 No. 8 bars. The steel is lumped at its centroid, 6 in. from the edge of the flange. The steel in the web and the compression steel are ignored, a conservative estimate.

Calculate effective flange width. Taken as the least of three values (8.10):

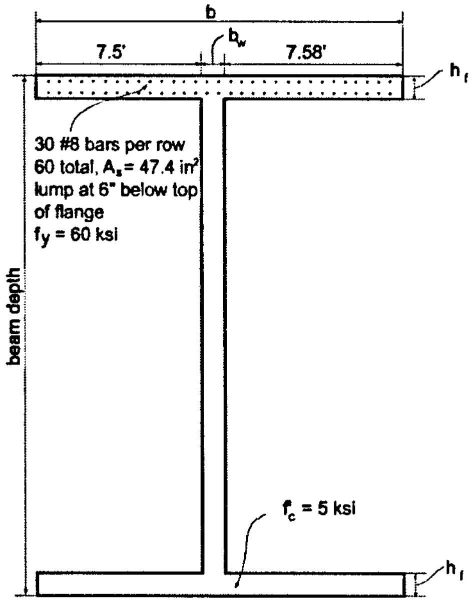
- one-quarter of the span length = ¼ × 135 ft = 33.75 ft
- sixteen times the thickness of the slab, plus the web thickness = 16 × 1 ft + 0.83 ft = 16.83 ft
- the average spacing of adjacent beams = 15.92 ft

Use 15.92 ft.

Calculate β_1, stress block factor:

$$\beta_1 = 0.85 - \frac{0.05 \cdot (f_c' - 30)}{7} = 0.85 - \frac{0.05 \cdot (34.5 - 30)}{7} = 0.82 \qquad (10.2.7.3)$$

The beam has the following properties:



Property	Non-SI Units	SI Units
A_s	47.4 in^2	30,573 mm^2
f_y	60 ksi	414 MPa
Beam depth	21 ft	6,405 mm
f_c	5 ksi	34.5 MPa
B	15.92 ft	4,856 mm
b_w	0.83 ft	253 mm
h_f	1 ft	305 mm
Cover	0.5 ft	153 mm
D	20.5 ft	6,253 mm

Assuming the section behaves as a rectangular beam, calculate c, distance between the neutral axis and the compressive face:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot \beta_1 \cdot b} = \frac{30{,}573 \cdot 414}{0.85 \cdot 34.5 \cdot 0.82 \cdot 4856} = 108\ mm < h_f$$

The assumption of behavior as a rectangular beam rather than a T-beam is correct because c is less than the h_f, and the compression zone does not extend into the web.

Calculate a, depth of equivalent stress block:

$$a = c \cdot \beta_1 = 108 \cdot 0.82 = 89\ mm$$

Find nominal flexural resistance:

$$M_n = A_s \cdot f_y \cdot (d - \frac{a}{2}) = 30{,}573 \cdot 414 \cdot (6253 - \frac{89}{2}) = 7.858 \cdot 10^{10}\ N{-}mm$$

Find factored flexural resistance:

$\varphi = 0.90$ for flexure and tension of reinforced concrete beams (9.3.2)

$$M_r = \varphi \cdot M_n = 0.9 \cdot 7.858 \cdot 10^{10} = 7.07 \cdot 10^{10}\ N{-}mm \qquad (9.3.2)$$

Check demand versus capacity:

$M_r > M_u$?

$70{,}700\ kN-m > 3{,}640\ kN-m \quad OK$

2) Check of shear in line 2, segment NP:

The cross section is the same as Example 1. Shear reinforcement consists of No. 7 stirrups spaced at 9 in.

Calculate A_v, steel area:

$$A_v = 0.60\ in^2 \cdot 2 = 1.20\ in^2$$

Note b_w, web width, and d, effective depth:

$$b_w = web\ thickness = 0.83\ ft$$
$$d = 20.5\ ft$$

For normal-density, reinforced concrete, φ, the resistance factor for shear, is 0.85 (9.3.2).

Property	***Non-SI Units***	***SI Units***
A_v	1.2 in^2	774 mm^2
f_c	5 ksi	34.5 MPa
S	0.75 ft	229 mm
b_w	0.83 ft	253 mm
D	20.5 ft	6,253 mm
φ	0.85	0.85

To find V_n, nominal shear resistance, compute:

$$V_n = V_c + V_s \tag{11.1}$$

Calculate first term:

$$V_c = \frac{\sqrt{f_c'}}{6} \cdot b_w \cdot d = \frac{\sqrt{34.5}}{6} \cdot 253 \cdot 6253 = 1{,}549\ kN \tag{11.3.1.1}$$

Calculate second term, assuming stirrups perpendicular to the axis of the member:

$$V_s = \frac{A_v \cdot f_y \cdot d}{s} = \frac{774 \cdot 414 \cdot 6253}{229} = 8{,}750\,kN \qquad (11.5.6.2)$$

Variable V_s cannot be taken greater than

$$V_{s\,\max} = (2/3) \cdot \sqrt{f_c^{'}} \cdot b_w \cdot d = (2/3) \cdot \sqrt{34.5} \cdot 253 \cdot 6253 = 6{,}196\,kN \quad (11.5.6.8)$$

Since 8,750 kN > 6,196 kN, use V_s = 6,196 kN.

$$V_n = V_c + V_s = 1{,}549\,kN + 6{,}196\,kN = 7{,}745\,kN$$

$$V_r = \varphi \cdot V_n = 0.85 \cdot 7{,}745\,kN = 6{,}583\,kN$$

Check demand versus capacity:

$V_r > V_u$?
$6{,}583\,kN > 863\,kN \quad OK$

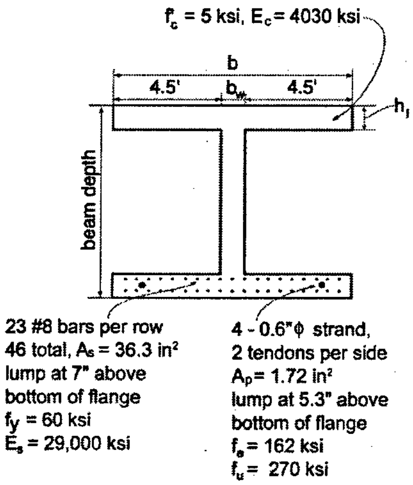
3) Check bending moment and crack width in prestressed beam - line 7, segment GH:

Calculate effective flange width. Taken as the least of three values (4.6.2.6.1):

- one-quarter of the effective span length = ¼ × 40 ft = 10 ft
- sixteen times the thickness of the slab, plus the web thickness = 16 × 1 ft + 1 ft = 17 ft
- the average spacing of adjacent beams = 16 ft

Use 10 ft.

The prestressed beam has the following properties:



Property	***Non SI Units***	***SI Units***
A_s	36.34 in^2	23,440 mm^2
f_y	60 ksi	414 MPa
E_s	29,000 ksi	2×10^5 MPa
Beam depth	8 ft	2,440 mm
f'_c	5 ksi	34.5 MPa
b	10 ft	3,050 mm
b_w	1.0 ft	305 mm
h_f	1.0 ft	305 mm
Cover	7 in	178 mm
d	7.42 ft	2,260 mm
A_{ps}	1.72 in^2	1,110 mm^2
f_{se}	162 ksi	1,117 MPa
f_{pu}	270 ksi	1,862 MPa
d_p	7.56 ft	2,303 mm

Calculate β_1, stress block factor:

$$\beta_1 = 0.85 - \frac{0.05 \cdot (f'_c - 30)}{7} = 0.85 - \frac{0.05 \cdot (34.5 - 30)}{7} = 0.82 \quad (10.2.7.3)$$

For low-relaxation strand or wire, use $f_{py} / f_{pu} = 0.90$ and $\gamma_p = 0.28$.

Assuming for the moment that the compression block remains within the flange:

$$\rho_p = \frac{A_{ps}}{b \cdot d_p} = \frac{1110}{3050 \cdot 2303} = 0.000158$$

$$\rho = \frac{A_s}{b \cdot d} = \frac{23440}{3050 \cdot 2260} = 0.0034$$

Given

$$f_{se} = 0.60 \cdot f_{pu} \geq 0.50 \cdot f_{pu}$$

and conservatively ignoring the contribution of compression reinforcement, the following approximate formula for bonded prestressing tendons can be used:

$$f_{ps} = f_{pu} \cdot \left\{1 - \frac{\gamma_p}{\beta_1} \cdot \left[\rho_p \cdot \frac{f_{pu}}{f_c'} + \frac{d}{d_p} \cdot \left(\rho \cdot \frac{f_y}{f_c'}\right)\right]\right\}$$

$$= f_{pu} \cdot \left\{1 - \frac{0.28}{0.82} \cdot \left[0.000158 \cdot \frac{1862}{34.5} + \frac{2260}{2303} \cdot (0.0034 \cdot 12)\right]\right\} = 0.98 \cdot f_{pu} \qquad (18.7.2)$$

$$= 0.98 \cdot 1862 = 1831\, Mpa$$

Check that a, the depth of equivalent stress block, does fall entirely within the flange:

$$a = \frac{A_{ps} \cdot f_{ps} + A_s \cdot f_y}{0.85 \cdot f_c' \cdot b} = \frac{1110 \cdot 1831 + 23440 \cdot 414}{0.85 \cdot 34.5 \cdot 3050} = 131 mm < h_f$$

Hence, it is appropriate to analyze this section as a rectangular beam.

Find nominal flexural resistance:

$$M_n = A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) + A_{ps} \cdot f_{ps}\left(d_p - \frac{a}{2}\right)$$

$$= 23{,}440 \cdot 414 \cdot \left(2260 - \frac{131}{2}\right) + 1110 \cdot 1831\left(2303 - \frac{131}{2}\right)$$

$$= 2.58x10^{10}\, N - mm$$

Find factored flexural resistance:

$\varphi = 0.9$ for flexure and tension of prestressed concrete

$$M_r = \varphi \cdot M_n = 0.9 \cdot 2.58 \cdot 10^{10} = 23{,}220\ kN\!-\!m$$

Check demand versus capacity:

$M_r > M_u$?

$23{,}220\, kN - m$ >2422 $kN\!-\!m$ *OK*

The moment capacity is sufficient. Next, a check of cracking is performed to ensure that there is no cracking or small cracks in the base of the slab due to bending moment.

To calculate the stress in the bottom-most layer of deformed reinforcement, the stress in the concrete at the location of the centroid of the rebar is first found by

$$\sigma_{concrete\ total} = \sigma_d + \sigma_{pa} + \sigma_{pb}$$

The tensile stress due to the unfactored bending moment demand is

$$\sigma_d = \frac{M_{svc}/1.7 \cdot y}{I} = \frac{2.42 \cdot 10^9 / 1.7 \cdot 1042}{2.29 \cdot 10^{12}} = 0.65 MPa$$

where

y = distance between the centroids of the concrete section and the deformed reinforcement = 1,042 mm
I = moment of inertia of the section = 2.29×10^{12} mm^4

The compressive stress due to the axial force applied by the prestressing tendons is

$$\sigma_{pa} = \frac{-P}{A} = \frac{-A_{ps} \cdot f_{se}}{A} = \frac{-1110 \cdot 1117}{2.42 \cdot 10^6} = -0.51 MPa$$

where

P = axial force applied by the prestressing tendons = 1,240 kN
A = area of the section = 2.42×10^6 mm^2

The compressive stress due to the bending moment applied by the prestressing force is

$$\sigma_{pb} = \frac{-P \cdot e \cdot y}{I} = \frac{-1.24 \cdot 10^6 \cdot 1085.3 \cdot 1042}{2.29 \cdot 10^{12}} = -0.61 MPa$$

where

e = distance from the prestressing tendons to the centroid of the section
$\sigma_{concrete\ total}$ = 0.65 – 0.51 – 0.61 = -0.47 MPa (compression)

Note that, by inspection, the entire concrete section is in compression at service loads.

$$\sigma_{rein forcing} = \sigma_{concrete} \cdot E_c / E_s = -0.47/7.2 = -0.065 MPa$$

Thus, because there is no tension in the reinforcing or concrete of the section, there will be no cracking. ACI 10.6.4 indicates that no maximum reinforcement spacing requirements need be met.

Design Example 3

The example calculation performed below is intended to make a comparison among the hydraulic factor design in EM 1110-2-2104, allowable stress criteria for crack control, and the new crack control provision in ACI 318M-99.

A water-retaining reinforced concrete abutment wall is subject to hydrostatic loading on one side. The wall is designed using the hydraulic factor design

method in EM 1110-2-2104. Then, the steel stress and crack width under the service condition are calculated. In addition, the spacing of reinforcing steel bars is checked using ACI 318M-99, "Building Code Requirements for Structural Concrete and Commentary."

All section numbers refer to sections of ACI 318M-99 unless otherwise noted. All SI units are in newtons and millimeters, and non-SI in pounds and inches unless otherwise noted.

Demand calculations

The wall height and hydrostatic head are both taken to be 4,880 mm (16 ft, 0 in.). The wall thickness is 610 mm (2 ft, 0 in.). Concrete strength f'_c is 34.5 MPa (5,000 psi), and steel f_y is 414 MPa (60,000 psi).

The load demand in both moment and shear force are calculated over a unit length of wall. The load demand is modified by a load factor of 1.7 and a hydraulic factor of 1.3 for all moments and shears to determine ultimate loads. Strength design of the wall is based on these EM 1110-2-2104 requirements:

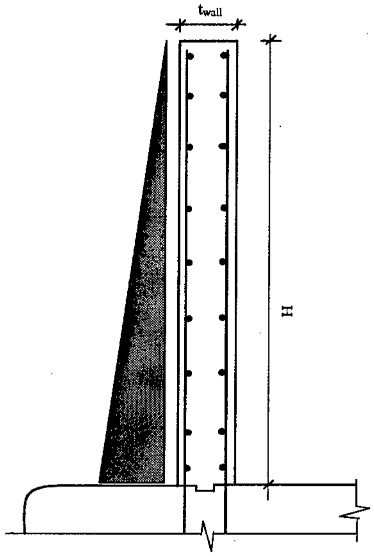
- Flexural strength demand:
  M_u = 418.7 kN-m per meter length of wall

- Shear strength demand:
  V_u = 257.4 kN per meter length of wall

Capacity calculations

The flexural strength and shear capacity are checked against the demands at critical sections using the hydraulic factor method. The following tabulated data assume No. 10 bars at 300 mm (12 in.) with 100 mm (4 in.) concrete cover. As a conservative simplification, compression reinforcement is ignored in the calculations.

The cantilever wall has the following properties:



Property	SI Units
Height	4,880
t_{wall}	610
L_{wall}	1,000
C_c (cover)	101.7
Diam bar	32.3
Abar	819
Spacing	305
B	1,000
D (effective depth)	492.2
A_s	2,687
$f'c$	34.5
f_y	414

First, the effective depth is found from

$$d = t_{wall} - C_c - \frac{\phi_{rebar}}{2} = 610 - 101.7 - \frac{32.3}{2} = 492.2\,mm$$

Calculate β_1, stress block factor:

$$\beta_1 = 0.85 - \frac{0.05 \cdot (f_c' - 30)}{7} = 0.85 - \frac{0.05 \cdot (34.5 - 30)}{7} = 0.82 \qquad (10.2.7.3)$$

Treating the section as a rectangular beam, calculate c, distance between the neutral axis and the compressive face:

$$c = \frac{A_s \cdot f_y}{0.85 \cdot f_c' \cdot \beta_1 \cdot b} = \frac{2687 \cdot 414}{0.85 \cdot 34.5 \cdot 0.82 \cdot 1000} = 46.37\ mm$$

Calculate a, depth of equivalent stress block:

$$a = c \cdot \beta_1 = 46.37 \cdot 0.82 = 37.93\ mm$$

Find nominal flexural resistance:

$$M_n = A_s \cdot f_y \cdot (d - \frac{a}{2}) = 2687 \cdot 414 \cdot (492.2 - \frac{37.93}{2}) = 5.263 \cdot 10^8\ N\text{–}mm$$

Find factored flexural resistance:

$$\varphi = 0.90 \text{ for flexure and tension of reinforced concrete beams} \qquad (9.3.2)$$

$$M_r = \varphi \cdot M_n = 0.9 \cdot 5.263 \cdot 10^8 = 4.737 \cdot 10^8 \; N\text{–}mm \qquad (9.3.2)$$

Check demand versus capacity:

$M_r > M_u$?
$473.7 \; kN\text{–}m > 418.7 \; kN\text{–}m \quad OK$

Next, check the shear strength at the base of the wall. The nominal shear resistance is computed as follows:

$$V_n = V_c + V_s$$

Since $V_s = 0$,

$$V_n = V_c = \frac{\sqrt{f_c'}}{6} \cdot b_w \cdot d = \frac{\sqrt{34.5}}{6} \cdot 1000 \cdot 492.2 = 481.8 \; kN \qquad (11.3.1.1)$$

The resistance factor for shear, φ, is 0.85 for normal-density, reinforced concrete (9.3.2).

$$V_r = \varphi \cdot V_n = 0.85 \cdot 481.8 \, kN = 409.5 \, kN$$

Check demand versus capacity:

$V_r > V_u$?
$6{,}583 \, kN > 863 \, kN \quad$ *OK, minimum reinforcement may be required*

Steel stresses and crack width

The stress in the steel under service loads is calculated, considering a cracked, elastic section. Then, crack widths are derived based on the Gergely-Lutz equation:

$$\rho = \frac{A_s}{b \cdot d} = \frac{2687}{1000 \cdot 492.2} = 0.0055$$

$$\rho_{balanced} = 0.85 \cdot \beta_1 \frac{f_c'}{f_y} \frac{\varepsilon_u}{\varepsilon_u + f_y / E_s} = 0.85 \cdot 0.82 \frac{34.5}{414} \frac{0.003}{0.003 + 414/200{,}000} = 0.0343$$

Minimum and maximum reinforcement ratios are shown in the table. Note that ρ may not be more than 50 percent of ρ_b per EM 1110-2-2104. This is one of the criteria that negate the need for a calculation of cracking according to the EM.

To find the stress:

$$n = \frac{E_s}{E_c} = \frac{200{,}000}{27{,}800} = 7.2$$

$$k = \sqrt{(\rho n)^2 + 2 \cdot \rho n} - \rho n = \sqrt{(0.0393)^2 + 2 \cdot 0.0393} - 0.393 = 0.244$$

$$jd = \left(1 - \frac{k}{3}\right) \cdot d = \left(1 - \frac{0.244}{3}\right) \cdot 492.2 = 452.2\ mm\ \text{(18 in.)}$$

$$f_s = \frac{M_{service}}{A_s \cdot jd} = \frac{4.187 \cdot 10^8 / (1.3 \cdot 1.7)}{2687 \cdot 452.2} = 156\ MPa\ \text{(22 ksi)}$$

Design data for the configuration with No. 10 bars at 300 mm (12 in.) and 100 mm (4 in.) cover are summarized as follows:

β_1	0.82		**ρ_{max} = 0.50 ρbal**	0.0171
c	46.37		***N***	7.2
a	37.93		ρ_n	0.0393
V_n	481,788		***K***	0.244
M_n	526,292,785		***Jd***	452.2
0.85V_n	409,520		f_s	156
0.9M_n	473,663,506		β	1.32
ρ	0.0055		d_c	117.85
ρ_{min} (as a beam)	0.0035		***A***	71889
ρ_{bal}	0.0343		***W***	0.461

The crack width calculation is as follows:

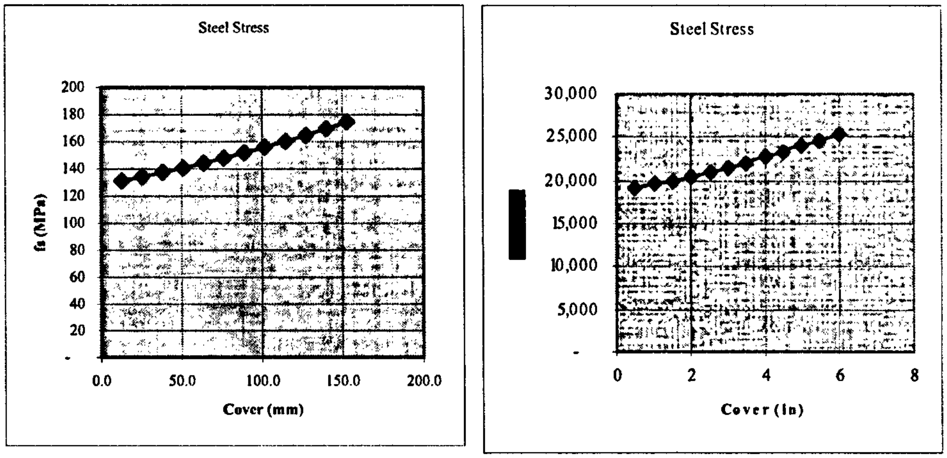
$$\beta = \frac{t_{wall} - kd}{d - kd} = \frac{610 - 0.244 \cdot 492.2}{492.2 - 0.244 \cdot 492.2} = 1.32$$

$$d_c = t_{wall} - d = 610 - 492.2 = 117.85\ mm\ \text{(4.7 in)}$$

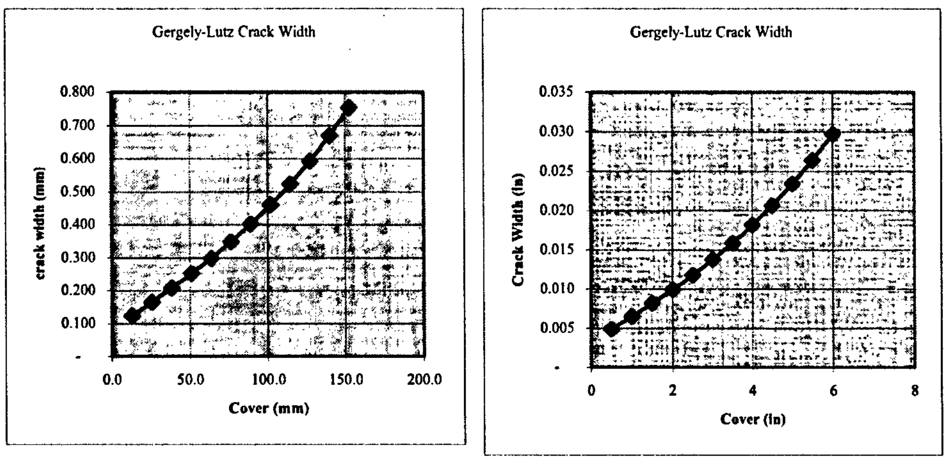
$$A = 2 \cdot d_c \cdot bar\ spacing = 2 \cdot 117.85 \cdot 305 = 71{,}889\ mm^2\ \text{(115 in}^2\text{)}$$

$$width = \left[0.001 \cdot 0.076 \cdot \beta \cdot (f_s * 0.145) \cdot \sqrt[3]{d_c \cdot A / 16{,}393.75}\right] \cdot 305/12 =$$
$$\left[0.001 \cdot 0.076 \cdot 1.32 \cdot (156 * 0.145) \cdot \sqrt[3]{117.85 \cdot 71{,}889/16{,}393.75}\right] \cdot 305/12 = 0.461\ mm\ (0.018in)$$

With the above calculation procedures, steel stresses and crack width are calculated with varying concrete cover. The results are plotted in the following charts.



(SI Units) (Non-SI Units)
Steel Stress Variation with Concrete Cover



(SI Units) (Non-SI Units)
Crack Width at Concrete Surface with Concrete Cover

Design Example 4

This example considers floating stability of a float-in concrete segment as described in Example 1. The process used for performing the calculations in this example is described in Chapter 5. The tainter gate segment is 333.0 ft (101.5 m) in length and 104.5 ft (31.9 m) in width. Figure E.5 illustrates the float-in segment in the process of floating out of the dry dock. Figure E.6 shows the configuration and dimensions of the segment.

In general, to check the ballasting and floating stability, there are four stages for the segments:

- Segment ballasting and deballasting in dock.
- Trim segments for float-out from dock to outfitting pier.
- Trim segments for float-out from outfitting pier to immersion site.

- Ballast segments down onto caissons.

This example shows the procedure of checking floating stability of a precast concrete dam segment during the process of transporting from a precast dry dock to an outfitting site (stage a).

The segments will be towed from the dock to the outfitting pier using tugs. The maximum segment draft during this transport is 10 ft. To be able to float out the segments during low water, the maximum weight of the segments may not exceed the calculated weight by more than 3 percent. No minimum transport weight is required. The metacentric height, *GM*, corrected for free water surfaces shall not be less than 4 ft for any direction of inclination.

The following formula is used to calculate *GM*:

$$GM = \frac{I}{V} + KB - KG$$

where

GM = metacentric height (ft or m)
KB = height of center of buoyancy (ft or m)
KG = height of center of gravity (ft or m)
I = moment of inertia of water plane (ft^4 or m^4)
V = volume of displacement (ft^3 or m^3)

Detailed calculations are shown in Table E.1 for the stage of initial floating-out from the casting dock to the outfitting pier. Stability calculations are summarized as follows:

Weight of the segment W = 22772 kips (101 MN)

$$V_z M = \sum V_{zi} W_i = 191689\ k-ft \text{ (260 MN-m)}$$

$$KG = \frac{V_z M}{W} = \frac{191689}{22772} = 8.4\ ft \text{ (2.56 m)}$$

$$\text{Volume of displacement } V = \frac{W}{\rho} = \frac{22772}{0.0624} = 364936\ ft^3 \text{ (10342 m}^3\text{)}$$

$$Draft\ D = \frac{V}{bl} = \frac{364936}{104.5 * 333} = 10.5\ ft \text{ (3.2 m)}$$

KB = D/2 = 5.25 ft (1.6 m)

$$I_{transverse} = \frac{lb^3}{12} = \frac{333 * 104.5^3}{12} = 31667360\ ft^4 \text{ (273600 m}^4\text{)}$$

$$BM = \frac{I_{transverse}}{V} = \frac{31667369}{364936} = 86.77\ ft\ \ (26.5\ \text{m})$$

$$GM = \frac{I}{V} + KB - KG = 86.77 + 5.25 - 8.4 = 83.62\,ft\ \ (25.5\ \text{m})$$

$GM > 4$ ft (1.2 m) **OK**

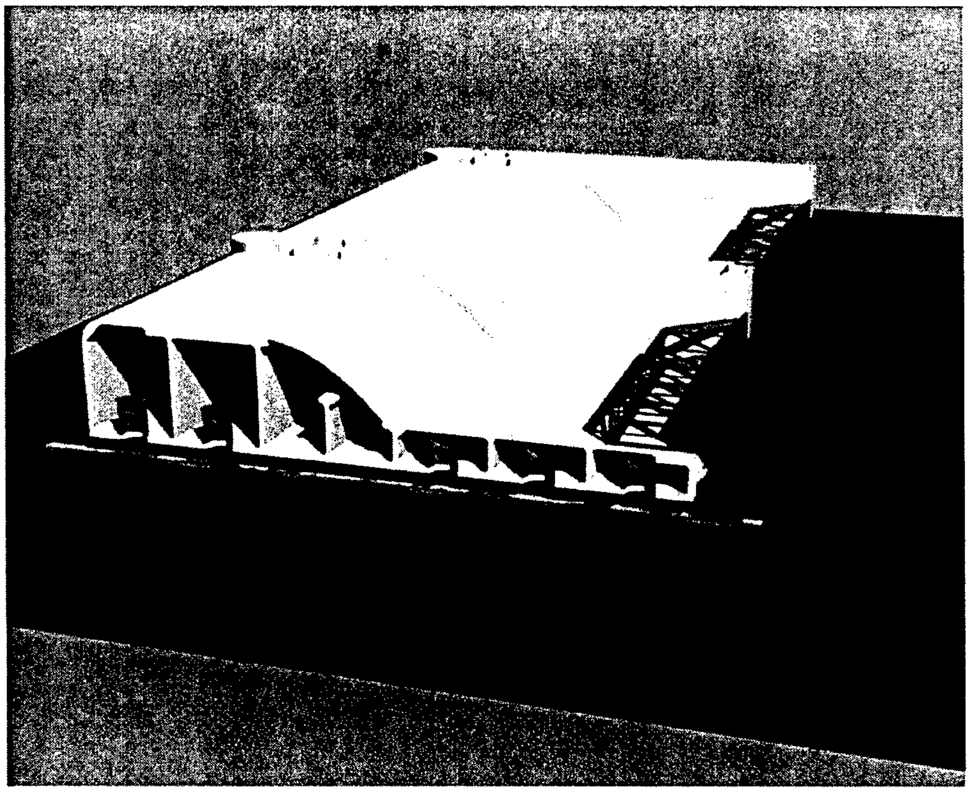


Figure E.5. A precast dam segment launching out of a dry dock

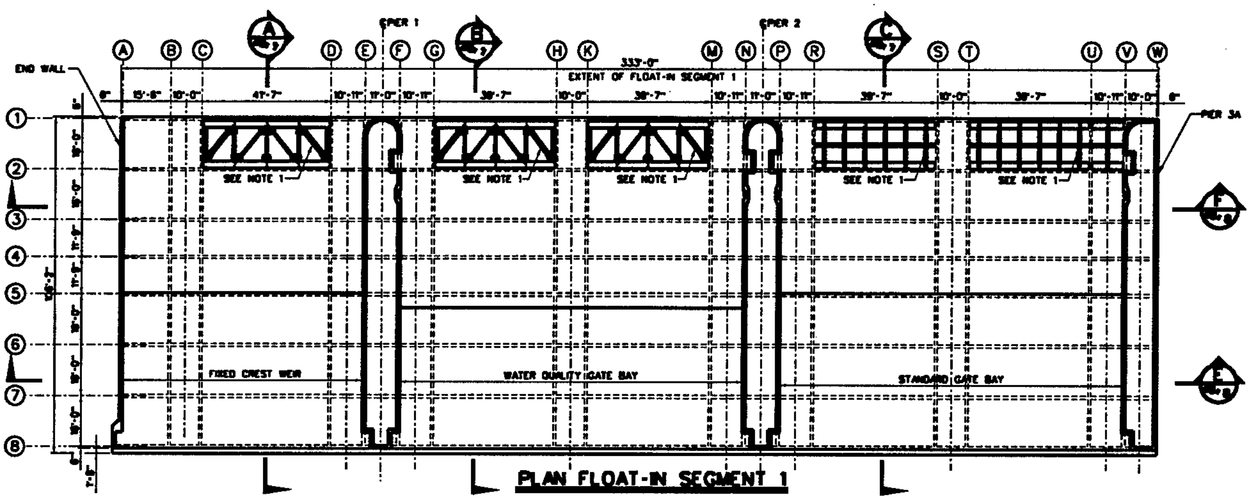


Figure E.6. Plan of float-in segment 1

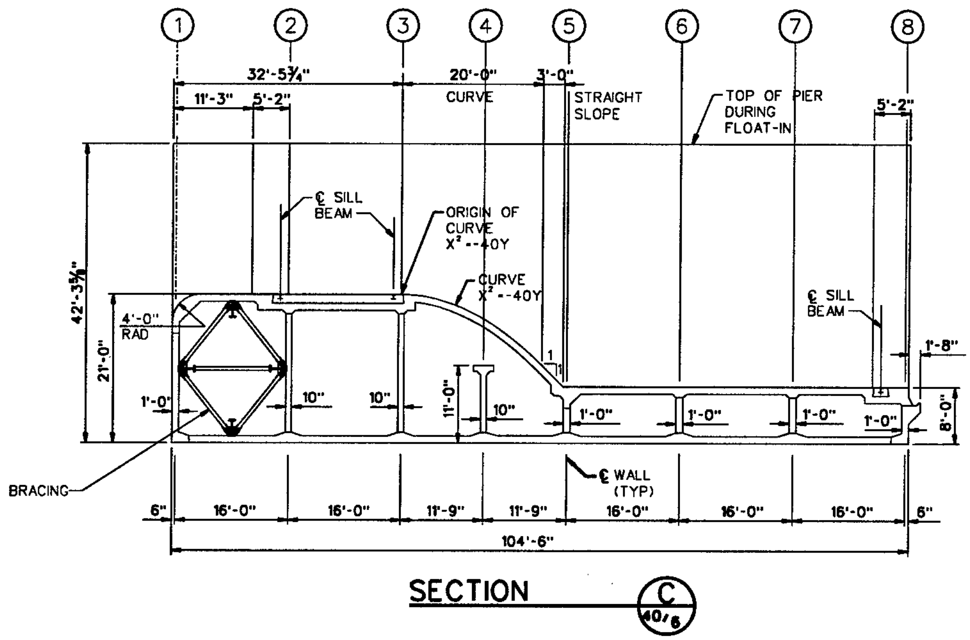


Figure E.7. Section C of float-in segment 1

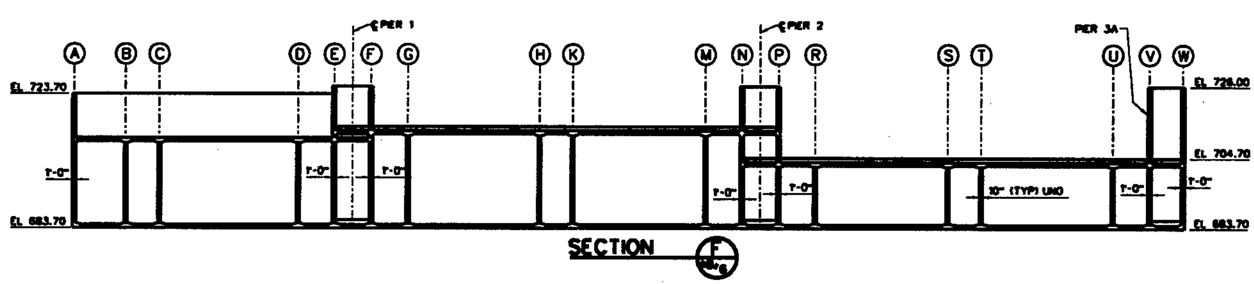


Figure E.8. Section F of float-in segment 1

Case: Float-Out

All top slabs installed except over two upstream compartments at FCW only
All external walls 12 inches
All internal transverse walls 10 inches - except pier walls
All internal transverse constructed by light weight concrete - except pier walls
All other internal & external walls at final height
Caisson openings (60-inch diameter) with 6'-0" tall corrugated pipe sleeves filled w/ water - covered
Baffle blocks not installed
Temporary d/s bulkhead 6 ft (one segment)
6-feet concrete added in some downstream compartments

Item	Longitudinal Dimension [ft]	Transverse Dimension [ft]	Vertical Dimension [ft]	Area [ft2]	Density [k/cft]	Quantity	Weight [k]	VzCG [ft]	VxCG [ft]	VyCG [ft]	VzW [k-ft]	VxW [k-ft]	VyW [k-ft]
	"x"	"y"	"z"										
Bottom Slab and Internal Transverse Walls FCW	84.00	-	-	170.85	0.1400	1	2,009	3.44	124.50	-4.71	6,912	250,145	-9,463
Bottom Slab and Internal Transverse Walls WQB	122.00	-	-	182.39	0.1400	1	3,115	4.52	21.50	-5.45	14,081	66,977	-16,978
Bottom Slab and Internal Transverse Walls SGB	127.00	-	-	165.39	0.1400	1	2,941	2.78	-100.50	-3.44	8,175	-295,534	-10,116
Bottom Slab Openings - Row "BC"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	145.50	0.00	-7	-2,000	0
Bottom Slab Openings - Row "DE"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	93.50	0.00	-7	-1,285	0
Bottom Slab Openings - Row "FG"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	71.50	0.00	-7	-983	0
Bottom Slab Openings - Row "HJ"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	21.50	0.00	-7	-296	0
Bottom Slab Openings - Row "KL"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	-28.50	0.00	-7	392	0
Bottom Slab Openings - Row "MN"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	-50.50	0.00	-7	694	0
Bottom Slab Openings - Row "PR"	5.00	5.00	1.0	-	-0.1400	5	-14	0.50	-100.50	0.00	-7	1,381	0
Bottom Slab Openings - Row "ST"	5.00	5.00	1.0	-	-0.1400	3	-8	0.50	-150.50	0.00	-4	1,241	0
Bottom Slab Openings - Row "TW"	5.00	5.00	1.0	-	-0.1400	3	-8	0.50	-160.50	0.00	-4	1,324	0
Bottom Slab Openings - Row "12"	5.00	5.00	1.0	-	-0.1400	8	0	0.50	0.00	-43.75	0	0	962
Bottom Slab Openings - Row "23"	5.00	5.00	1.0	-	-0.1400	8	0	0.50	0.00	-27.75	0	0	153
Bottom Slab Openings - Row "34"	5.00	5.00	1.0	-	-0.1400	1	0	0.50	0.00	-13.75	0	0	76
Bottom Slab Openings - Row "56"	5.00	5.00	1.0	-	-0.1400	8	0	0.50	0.00	11.75	0	0	-65
Bottom Slab Openings - Row "67"	5.00	5.00	1.0	-	-0.1400	8	0	0.50	0.00	27.75	0	0	-153
Bottom Slab Openings - Row "78"	5.00	5.00	1.0	-	-0.1400	8	0	0.50	0.00	43.75	0	0	-962
Top Slab and downstream wall - FCW	82.00	-	-	90.92	0.1600	1	1,193	8.98	127.50	17.83	10,712	152,091	21,269
Upstream Wall - FCW	84.00	-	-	22.00	0.1600	1	296	12.30	124.50	-51.55	3,637	36,812	-15,242
Top Slab and Up - and Downstream Walls - WQB	118.00	-	-	175.00	0.1600	1	3,304	15.72	21.50	-4.33	51,939	71,036	-14,306
Top Slab and Up - and Downstream Walls - SGB	118.00	-	-	154.85	0.1600	1	2,924	12.04	-100.50	-1.92	35,200	-293,819	-5,613
Transverse External Wall - FCW - A	-	-	19.0	113.30	0.1600	1	344	11.00	166.10	-3.58	3,789	57,210	-1,233
Transverse Wall Panel "1" - FCW -Internal B	0.83	-	-	295.70	0.1400	1	34	10.87	150.50	-43.71	373	5,171	-1,502
Transverse Wall Panel "2"	0.83	-	-	298.70	0.1400	1	35	10.91	150.50	-27.72	379	5,224	-962
Transverse Wall Panel "3"	0.83	-	-	149.80	0.1400	1	17	8.15	150.50	-14.43	142	2,620	-251
Transverse Wall Panel "4"	0.83	-	-	74.90	0.1400	1	9	4.58	150.50	-2.56	40	1,310	-22
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	150.50	11.75	42	1,556	122
Transverse Wall Panel "6"	0.83	-	-	89.00	0.1400	1	10	4.02	150.50	27.75	42	1,556	287
Transverse Wall Panel "7"	0.83	-	-	89.00	0.1400	1	10	4.02	150.50	43.75	42	1,556	452
Transverse Wall Panel "1" - FCW -Internal C	0.83	-	-	295.70	0.1400	1	34	10.87	140.50	-43.71	373	4,828	-1,502
Transverse Wall Panel "2"	0.83	-	-	298.70	0.1400	1	35	10.91	140.50	-27.72	379	4,877	-962
Transverse Wall Panel "3"	0.83	-	-	149.80	0.1400	1	17	8.15	140.50	-14.43	142	2,446	-251
Transverse Wall Panel "4"	0.83	-	-	74.90	0.1400	1	9	4.58	140.50	-2.56	40	1,223	-22
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	140.50	11.75	42	1,453	122
Transverse Wall Panel "6"	0.83	-	-	89.00	0.1400	1	10	4.02	140.50	27.75	42	1,453	287
Transverse Wall Panel "7"	0.83	-	-	89.00	0.1400	1	10	4.02	140.50	43.75	42	1,453	452
Transverse Wall Panel "1" - FCW -Internal D	0.83	-	-	295.70	0.1400	1	34	10.87	98.50	-43.71	373	3,384	-1,502
Transverse Wall Panel "2"	0.83	-	-	298.70	0.1400	1	35	10.91	98.50	-27.72	379	3,419	-962
Transverse Wall Panel "3"	0.83	-	-	149.80	0.1400	1	17	8.15	98.50	-14.43	142	1,715	-251
Transverse Wall Panel "4"	0.83	-	-	74.90	0.1400	1	9	4.58	98.50	-2.56	40	857	-22
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	98.50	11.75	42	1,019	122
Transverse Wall Panel "6"	0.83	-	-	89.00	0.1400	1	10	4.02	98.50	27.75	42	1,019	287
Transverse Wall Panel "7"	0.83	-	-	89.00	0.1400	1	10	4.02	98.50	43.75	42	1,019	452
Pier 1	-	-	21.8	230.80	0.1600	1	803	11.00	82.50	-0.22	8,835	66,263	-175
Transverse Wall Panel "1" - WQB - Internal Wall G	0.83	-	-	415.65	0.1400	1	48	14.85	66.50	-43.71	717	3,212	-2,111
Transverse Wall Panel "2"	0.83	-	-	409.20	0.1400	1	48	14.54	66.50	-27.75	691	3,162	-1,319
Transverse Wall Panel "3"	0.83	-	-	448.30	0.1400	1	52	12.00	66.50	-9.76	625	3,464	-508
Transverse Wall Panel "4"	0.83	-	-	87.15	0.1400	1	10	3.96	66.50	11.83	40	673	120
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	66.50	27.75	42	688	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	66.50	43.39	37	641	418
Transverse Wall Panel "1" - WQB - Internal Wall H	0.83	-	-	415.65	0.1400	1	48	14.85	26.50	-43.71	717	1,280	-2,111
Transverse Wall Panel "2"	0.83	-	-	409.20	0.1400	1	48	14.54	26.50	-27.75	691	1,260	-1,319
Transverse Wall Panel "3"	0.83	-	-	448.30	0.1400	1	52	12.00	26.50	-9.76	625	1,380	-508
Transverse Wall Panel "4"	0.83	-	-	87.15	0.1400	1	10	3.96	26.50	11.83	40	268	120
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	26.50	27.75	42	274	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	26.50	43.39	37	256	418

Table E1. Stability checking for precast dam segment 1 (Sheet 1 of 3)

Item	Longitudinal Dimension [ft]	Transverse Dimension [ft]	Vertical Dimension [ft]	Area [ft2]	Density [k/cft]	Quantity	Weight [k]	VzCG [ft]	VxCG [ft]	VyCG [ft]	VzW [k-ft]	VxW [k-ft]	VyW [k-ft]
	"x"	"y"	"z"										
Transverse Wall Panel "1" - WQB - Internal Wall J	0.83	-	-	415.65	0.1400	1	48	14.85	16.50	-43.71	717	797	-2,111
Transverse Wall Panel "2"	0.83	-	-	409.20	0.1400	1	48	14.54	16.50	-27.75	691	785	-1,319
Transverse Wall Panel "3"	0.83	-	-	448.30	0.1400	1	52	12.00	16.50	-9.76	625	860	-508
Transverse Wall Panel "4"	0.83	-	-	87.15	0.1400	1	10	3.96	16.50	11.83	40	167	120
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	16.50	27.75	42	171	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	16.50	43.39	37	159	418
Transverse Wall Panel "1" - WQB - Internal Wall K	0.83	-	-	415.65	0.1400	1	48	14.85	-23.50	-43.71	717	-1,135	-2,111
Transverse Wall Panel "2"	0.83	-	-	409.20	0.1400	1	48	14.54	-23.50	-27.75	691	-1,117	-1,319
Transverse Wall Panel "3"	0.83	-	-	448.30	0.1400	1	52	12.00	-23.50	-9.76	625	-1,224	-508
Transverse Wall Panel "4"	0.83	-	-	87.15	0.1400	1	10	3.96	-23.50	11.83	40	-238	120
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	-23.50	27.75	42	-243	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	-23.50	43.39	37	-227	418
Pier 2	-	-	24.0	237.55	0.1600	1	912	9.50	-39.50	-0.79	8,666	-36,032	-718
Transverse Wall Panel "1" - Internal Wall N	0.83	-	-	275.40	0.1400	1	32	10.20	-55.50	-43.71	326	-1,776	-1,399
Transverse Wall Panel "2"	0.83	-	-	268.20	0.1400	1	31	9.89	-55.50	-27.75	308	-1,730	-865
Transverse Wall Panel "3"	0.83	-	-	307.70	0.1400	1	36	8.77	-55.50	-9.90	314	-1,984	-354
Transverse Wall Panel "4"	0.83	-	-	88.00	0.1400	1	10	3.99	-55.50	11.83	41	-568	121
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	-55.50	27.75	42	-574	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	-55.50	43.39	37	-535	418
Transverse Wall Panel "1" - Internal Wall P	0.83	-	-	275.40	0.1400	1	32	10.20	-95.50	-43.71	326	-3,056	-1,399
Transverse Wall Panel "2"	0.83	-	-	268.20	0.1400	1	31	9.89	-95.50	-27.75	308	-2,976	-865
Transverse Wall Panel "3"	0.83	-	-	307.70	0.1400	1	36	8.77	-95.50	-9.90	314	-3,415	-354
Transverse Wall Panel "4"	0.83	-	-	88.00	0.1400	1	10	3.99	-95.50	11.83	41	-977	121
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	-95.50	27.75	42	-988	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	-95.50	43.39	37	-921	418
Transverse Wall Panel "1" - Internal Wall R	0.83	-	-	275.40	0.1400	1	32	10.20	-105.50	-43.71	326	-3,376	-1,399
Transverse Wall Panel "2"	0.83	-	-	268.20	0.1400	1	31	9.89	-105.50	-27.75	308	-3,288	-865
Transverse Wall Panel "3"	0.83	-	-	307.70	0.1400	1	36	8.77	-105.50	-9.90	314	-3,772	-354
Transverse Wall Panel "4"	0.83	-	-	88.00	0.1400	1	10	3.99	-105.50	11.83	41	-1,079	121
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	-105.50	27.75	42	-1,091	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	-105.50	43.39	37	-1,018	418
Transverse Wall Panel "1" - Internal Wall S	0.83	-	-	275.40	0.1400	1	32	10.20	-145.50	-43.71	326	-4,656	-1,399
Transverse Wall Panel "2"	0.83	-	-	268.20	0.1400	1	31	9.89	-145.50	-27.75	308	-4,534	-865
Transverse Wall Panel "3"	0.83	-	-	307.70	0.1400	1	36	8.77	-145.50	-9.90	314	-5,202	-354
Transverse Wall Panel "4"	0.83	-	-	88.00	0.1400	1	10	3.99	-145.50	11.83	41	-1,488	121
Transverse Wall Panel "5"	0.83	-	-	89.00	0.1400	1	10	4.02	-145.50	27.75	42	-1,505	287
Transverse Wall Panel "6"	0.83	-	-	83.00	0.1400	1	10	3.85	-145.50	43.39	37	-1,403	418
Pier 3	-	-	19.5	228.50	0.1600	1	713	9.50	-161.50	-0.41	6,773	-115,137	-292
Temporary Bulkhead Downstream	316.00	0.20	6.0	-	0.5639	1	215	12.00	5.50	-48.00	2,580	1,183	-10,320
Adjustments													
Bracing in std. G.Bay							40	11.50	-105.50	-44.58	460	-4,220	-1,783
Bracing in WQ Bay							18	11.50	21.50	-44.58	207	387	-802
Grout Pipes							215	20.00	0.00	0.00	4,300	0	0
Post Tensioning							26	0.50	0.00	0.00	13	0	0
Pile Top Blockouts							67	4.00	0.00	0.00	268	0	0
Tow Fittings							6	30.00	0.00	0.00	180	0	0
Set Down Assemblies							84	2.50	-39.29	-0.83	210	-3,300	-70
#18 bars in piers							74	15.00	0.00	10.00	1,110	0	740
Openings for access in walls (88 Em)							-83	8.00	0.00	0.00	-664	0	0
Revised Weight of temporary bulkheads							28	12.00	0.00	52.00	336	0	1,456
Weight of thickened slab in piers							6	1.17	0.00	0.00	7	0	0
Weight of embedded plates							21	11.50	0.00	-43.70	242	0	-918

Table E1. (Sheet 2 of 3)

Item	Longitudinal Dimension [ft]	Transverse Dimension [ft]	Vertical Dimension [ft]	Area [ft2]	Density [k/cft]	Quantity	Weight [k]	VzCG [ft]	VxCG [ft]	VyCG [ft]	VzW [k-ft]	VxW [k-ft]	VyW [k-ft]
	"x"	"y"	"z"										
Ballast													
Ballast - Compartment B1	14.58	52.08	0.0	-	0.0624	1	0	0.00	158.50	-24.00	0	0	0
Ballast - Compartment B2	14.58	45.42	0.0	-	0.0624	1	0	0.00	158.50	27.75	0	0	0
Ballast - Compartment B3	59.92	15.08	0.0	-	0.0624	1	0	0.00	119.50	-43.75	0	0	0
Ballast - Compartment B4	59.92	37.00	0.0	-	0.0624	1	0	0.00	119.50	-16.00	0	0	0
Ballast - Compartment B4 - concrete BC-23	9.17	15.17	0.0	-	0.1600	1	0	0.00	145.50	-27.75	0	0	0
Ballast - Compartment B6 - concrete BC-78	9.17	15.08	0.0	-	0.1600	1	0	0.00	145.50	43.75	0	0	0
Ballast - Compartment B5	59.92	30.33	0.0	-	0.0624	1	0	0.00	119.50	19.75	0	0	0
Ballast - Compartment B6 - water	59.92	15.08	6.0	-	0.0624	1	338	4.00	119.50	43.75	1,353	40,434	14,803
Ballast - Compartment B6 - concrete DE-78	10.00	15.08	0.0	-	0.1600	1	0	0.00	93.50	43.75	0	0	0
Ballast - Compartment B7	10.00	15.08	0.0	-	0.0624	1	0	0.00	82.50	-43.75	0	0	0
Ballast - Compartment B8	10.00	37.00	0.0	-	0.0624	1	0	0.00	82.50	-16.00	0	0	0
Ballast - Compartment B9	10.00	30.33	0.0	-	0.0624	1	0	0.00	82.50	19.75	0	0	0
Ballast - Compartment B10	10.00	15.08	0.0	-	0.0624	1	0	0.00	82.50	43.75	0	0	0
Ballast - Compartment B11	48.75	15.08	0.0	-	0.0624	1	0	0.00	51.50	-43.75	0	0	0
Ballast - Compartment B12	48.75	37.00	0.0	-	0.0624	1	0	0.00	51.50	-16.00	0	0	0
Ballast - Compartment B13	48.75	30.33	0.0	-	0.0624	1	0	0.00	51.50	19.75	0	0	0
Ballast - Compartment B14 - concrete FG-78	10.00	15.08	6.0	-	0.1600	1	126	4.00	71.50	43.75	504	9,005	5,510
Ballast - Compartment B14 - concrete GH-78	38.75	15.08	2.8	-	0.1450	1	237	2.40	46.50	43.75	570	11,034	10,382
Ballast - Compartment B15	57.92	15.08	0.0	-	0.0624	1	0	0.00	-3.50	-43.75	0	0	0
Ballast - Compartment B16	57.92	37.00	0.0	-	0.0624	1	0	0.00	-3.50	-16.00	0	0	0
Ballast - Compartment B17	48.75	30.33	0.0	-	0.0624	1	0	0.00	-3.50	19.75	0	0	0
Ballast - Compartment B18 - water	57.92	15.08	6.0	-	0.0624	1	320	4.00	-3.50	43.75	1,279	-1,119	13,988
Ballast - Compartment B18 - concrete HJ-78	9.17	15.08	6.0	-	0.0976	1	69	4.00	21.50	43.75	278	1,494	3,039
Ballast - Compartment B18 - concrete JK-78	38.75	15.08	6.0	-	0.1450	1	508	4.00	-3.50	43.75	2,034	-1,780	22,247
Ballast - Compartment B18 - concrete KL-78	10.00	15.08	6.0	-	0.0976	1	77	4.00	-28.50	43.75	307	-2,190	3,361
Ballast - Compartment B19	10.00	15.13	0.0	-	0.0624	1	0	0.00	-39.50	-43.75	0	0	0
Ballast - Compartment B20	10.00	37.00	0.0	-	0.0624	1	0	0.00	-39.00	-16.00	0	0	0
Ballast - Compartment B21	10.00	30.33	0.0	-	0.0624	1	0	0.00	-39.00	19.75	0	0	0
Ballast - Compartment B22	10.00	15.08	0.0	-	0.0624	1	0	0.00	-39.00	43.75	0	0	0
Ballast - Compartment B23	48.75	15.08	0.0	-	0.0624	1	0	0.00	-70.00	-43.75	0	0	0
Ballast - Compartment B24	48.75	37.00	0.0	-	0.0624	1	0	0.00	-70.00	-16.00	0	0	0
Ballast - Compartment B25	48.75	30.33	0.0	-	0.0624	1	0	0.00	-70.00	19.75	0	0	0
Ballast - Compartment B26 - concrete MN-78	10.00	15.08	6.0	-	0.0976	1	77	4.00	-50.00	43.75	307	-3,841	3,361
Ballast - Compartment B26 - concrete NP-78	38.75	15.08	0.0	-	0.1400	1	0	0.00	-75.50	43.75	0	0	0
Ballast - Compartment B26 - water	48.75	15.08	6.0	-	0.0624	1	275	4.00	-70.00	43.75	1,101	-19,271	12,044
Ballast - Compartment B27 - water	57.92	15.08	0.0	-	0.0624	1	0	0.00	-124.50	-43.75	0	0	0
Ballast - Compartment B28	57.92	37.00	0.0	-	0.0624	1	0	0.00	-124.50	-16.00	0	0	0
Ballast - Compartment B29 - concrete PR-78	9.17	15.08	0.0	-	0.1600	1	0	0.00	-100.50	43.75	0	0	0
Ballast - Compartment B30 - concrete RS-78	38.75	15.08	0.0	-	0.1400	1	0	0.00	-125.00	43.75	0	0	0
Ballast - Compartment B30 - concrete ST-78	10.00	15.08	0.0	-	0.1600	1	0	0.00	-150.50	43.75	0	0	0
Ballast - Compartment B31	9.00	52.08	0.0	-	0.0624	1	0	0.00	-160.50	-24.00	0	0	0
Ballast - Compartment B32	9.00	45.42	0.0	-	0.1600	1	0	0.00	-160.50	27.75	0	0	0
sub-total							22,772				191,689	393	-1,076
Water density	k/cf	0.0624			Ballast Weight		2,089	kips					
Volume of displacement	cf	364,940											
Draft	ft	10.5											
KGz	ft	8.4											
BM - transverse	ft	86.8											
BM - longitudinal	ft	881.1											
KBz	ft	5.2											
GM -transverse	ft	83.6											
GM - longitudinal	ft	878.0											

Table E1. (Sheet 3 of 3)

Appendix F
Alternate Design Methods

Notation

$\mathbf{a}$ = distance along short side of a rectangular plate
$\mathbf{A_{cs}}$ = area of concrete in the strut
$\mathbf{A_g}$ = gross area of concrete section
$\mathbf{A_s}$ = area of reinforcing steel
$\mathbf{A_{ss}}$ = area of parallel reinforcing in the strut
$\mathbf{b}$ = width of panel strip
$\mathbf{d}$ = depth from compression face to center of reinforcement
$\mathbf{f_c'}$ = specified concrete compressive strength
$\mathbf{f_{cu}}$ = limiting compressive stress of a strut
$\mathbf{f_y}$ = yield strength of reinforcing steel
$\mathbf{h}$ = height of panel strip
$\mathbf{M_u}$ = design moment
$\mathbf{M_x}$ = moment about the y axis
$\mathbf{M_y}$ = moment about the x-axis
$\mathbf{P_n}$ = nominal resistance of strut or tie
$\mathbf{P_r}$ = factored resistance of a strut or tie
$\mathbf{P_u}$ = factored/design axial load
$\mathbf{q}$ = uniform pressure
$\mathbf{R_u}$ = required coefficient of resistance
$\mathbf{T}$ = deformation loads
$\mathbf{w}$ = width of tributary area
$\mathbf{x}$ = distance along x-axis
$\boldsymbol{\beta'}$ = numerical factor
$\boldsymbol{\beta''}$ = numerical factor
$\boldsymbol{\alpha_s}$ = smallest angle between the compressive strut and adjoining tension ties
$\boldsymbol{\varepsilon_s}$ = tensile strain in the concrete in the direction of tension tie
$\boldsymbol{\phi}$ = resistance factor
$\boldsymbol{\rho}$ = reinforcement ratio
$\boldsymbol{\rho_b}$ = balanced reinforcement ratio
$\boldsymbol{\upsilon}$ = Poisson's ratio

Examples given in Appendix E are design methods that are certainly viable methods for designing precast elements associated with float-in and lift-in construction. However, there are, of course, other approaches that can be taken when performing designs. This appendix provides some brief descriptions of other approaches that can be taken during the design of precast thin-wall panels for float-in and lift-in construction.

Strut-Tie Modeling

For non-Bernoulli structural regions, where plane sections do not remain plane after deformation, including areas of discontinuity in geometry and of concentrated loading, the member capacity may be determined using force models with such elements as struts, ties, nodes, stress fields, or similar elements that satisfy equilibrium conditions. For many non-Bernoulli members, such as corbels, simple deep beams, and post-tensioning anchorages, the equilibrium force models are well understood and may be used with confidence. For more complex non-Bernoulli members, such as continuous deep beams with openings, or intersections of multiple sections, determining the appropriate equilibrium force model may require the evaluation of stress trajectories determined from the theory of elasticity, elastic finite element models, nonlinear finite element models, or experimental models.

The force model developed should be compatible with the internal stress fields and deformation conditions expected within the member for the ultimate load condition. It is noted that, for a given member geometry, the internal stress fields and ultimate member capacity can vary significantly depending on such factors as boundary conditions, loading patterns, reinforcing patterns, and concrete behavior as influenced by tensile fields, passive confinement, or active confinement. Furthermore, the force model shall address any tensile forces induced by deviations in the compressive fields under consideration.

Guidance for the development of specific strut and tie models can be found in the literature (Schlaich, Schafer, and Jennewein 1987; Collins and Mitchell 1991), including ACI's Recommended Practice. However, for consistency with the AASHTO guidelines given in Chapter 10, the following guidance uses AASHTO's 1994 LRFD approach.

The factored resistance, P_r, of a strut or tie shall be taken as $P_r = \phi P_n$, where

P_n = nominal resistance of strut or tie, as discussed below
ϕ = resistance factor, as discussed below

- When calculating the factored capacity of struts and ties, the resistance factor for a compressive strut should be 0.7, and the resistance factor for a tie should be 0.9.

- The nominal capacity of a compressive strut should be taken as the effective cross-sectional area of the strut times the limiting compressive strength of the strut.

- The effective cross-sectional area of the strut shall be determined considering the anchorage conditions for the strut and the geometry of the member.

The nominal resistance of an unreinforced compressive strut may be taken as

$$P_n = f_{cu} * A_{cs}$$

where

P_n = nominal resistance of a compressive strut
f_{cu} = limiting compressive stress of a strut, as given below

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \le 0.85 f'_c$$

$$\varepsilon_1 = (\varepsilon_s + 0.002)\cot^2 \alpha_s$$

where

α_s = smallest angle between the compressive strut and adjoining tension ties
ε_s = tensile strain in the concrete in the direction of tension tie
f'_c = specified concrete compressive strength

For compressive struts with reinforcing steel parallel to the direction of the strut and which has been detailed to develop its yield stress in compression,

$$P_n = f_{cu} * A_{cs} + f_y * A_{ss}$$

where A_{ss} is the area of parallel reinforcing steel in the strut.

The concrete compressive stress in the node regions of the strut shall not exceed the following, unless confining reinforcement is provided and its effect is supported by experimentation:

- For nodes bounded by compressive struts and bearing areas: $0.85\phi f'_c$
- For nodes anchoring a one-direction tension tie: $0.75\phi f'_c$
- For nodes anchoring tension ties in more than one direction: $0.65\phi f'_c$

Plate Theory

The title of this report suggests that plate theory might be applicable (i.e. ". . . Thin-Wall Panels . . .") for the types of structures being considered. Three types of plates can be distinguished as

- thin plates with small deflections.
- thin plates with large deflections.
- thick plates.

Thin plate theory assumes that the deflections of the plate are small in comparison to the thickness of the plate. Likewise, bending of the plate due to lateral loads can be obtained using the following assumptions:

- There is no deformation in the middle plane of the plate. This plane remains neutral during bending.
- Points of the plate lying initially on a normal-to-the-middle plane of the plate remain on the normal-to-the-middle surface of the plate after bending.
- The normal stresses in the direction transverse to the plate can be disregarded.

While the equations for computing bending moments of plates are somewhat extensive, Timoshenko and Woinowksy-Krieger (1959) provide a number of solutions for different boundary conditions and loadings for thin plates. These can be adopted as needed in solving for bending stresses that may occur to thin-wall panels due to loading that is normal to the plate's surface.

For example, one of the most commonly used solutions is for a rectangular plate (Figure F.1) that is simply supported and uniformly loaded. For this particular case, the equation for the moment about the x-axis at $y = 0$, M_x, is given by

$$(Mx)y = 0 = \frac{qx(a-x)}{2} - qa^2\pi \sum_{m=1,3,5,\ldots}^{\infty} m^2 \left[2\nu B_m - (1-\nu)A_m\right]\sin\frac{m\pi x}{a} \tag{F-1}$$

where

q = uniform pressure
x = distance along x-axis
a = distance along short side of plate (Figure F.1)
m = summation counter
υ = Poisson's ratio

$$B_m = \frac{2}{\pi^5 m^5 \cosh\alpha_m}$$

$$A_m = -\frac{2(\alpha_m \tanh\alpha_m + 2)}{\pi^5 m^5 \cosh\alpha_m}$$

However, this equation can be rewritten as

$$(M_x)_{y=0} = \beta' qa^2 \tag{F-2}$$

where the values of β' (given in Table F.1) take into account all of the terms in Equation F-1 except for qa^2, and M_x can be computed by using Equation F-2 and Table F.1 by applying a given b/a and a distance x as a function of a.

Likewise, M_y, can be determined in a similar manner based on

$$(M_y)_{y=0} = \beta'' qa^2$$

where β_1' values are given in Table F.1.

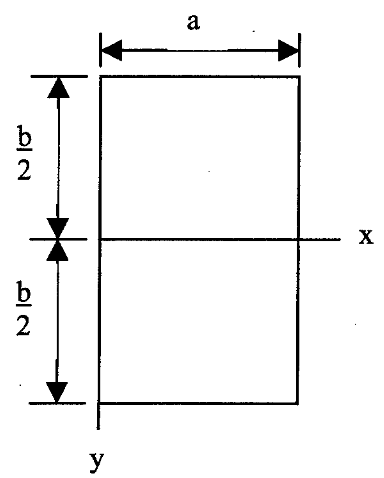


Figure F.1. Rectangular plate

As an example of how this can be used in design, consider the float-in section from the Braddock Dam Project. As can be seen in Figure E.7, if the panel size is based on the intermediate walls of the section, the maximum panel size is 16 by 39.58 ft. This results in a b/a ratio of 2.474, which gives a β' value of 0.11132 by interpolating between b/a values of 2.0 and 2.5.

For the transportation case, the float-in section will have a 10-ft draft; therefore, the pressure q will be

$$q = (10 \text{ ft})(62.5 \text{ lb/ft}^3) - (1 \text{ ft})(150 \text{ lb/ft}^3) = 475 \text{ lb/ft}^2$$

Using Equation F-2, M_x is given by

$$\begin{aligned} M_x &= 0.11132\,(475 \text{ lb/ft}^2)\,(16 \text{ ft})^2 \\ &= 13{,}537 \text{ ft-lb/ft } (12 \text{ in/ft}) \\ &= 162{,}438 \text{ in-lb/ft} = 162.4 \text{ in-k/ft} \end{aligned}$$

For the transportation case, the design moment, M_u, has a load factor of 1.4. Therefore,

$$\begin{aligned} M_u &= 1.4\,(M_x) = 1.4\,(162.4 \text{ in-k/ft}) \\ &= 227.4 \text{ in-k/ft} \end{aligned}$$

The required coefficient of resistance, R_u, is given by

$R_u = M_u(\mathrm{b/d}^2)$

Table F1. Numerical Factors for Use with Equation F-2

Numerical Factors β' and β1' for Bending Moments of Simply Supported Rectangular Plates Under Uniform Pressure q, v = 0.2, b ≥ a

b/a	Mx = β'qa2; y = 0					My = β1'qa2; y = 0				
	x = 0.1a	x = 0.2a	x = 0.3a	x = 0.4a	x = 0.5a	x = 0.1a	x = 0.2a	x = 0.3a	x = 0.4a	x = 0.5a
1.0	0.01975	0.03213	0.03936	0.04307	0.04420	0.01510	0.02751	0.03669	0.04231	0.04420
1.2	0.02456	0.04118	0.05168	0.05742	0.05924	0.01521	0.02777	0.03713	0.04290	0.04484
1.4	0.02873	0.04908	0.06249	0.07007	0.07252	0.01476	0.02697	0.03606	0.04168	0.04358
1.6	0.03220	0.05567	0.07153	0.08067	0.08366	0.01409	0.02568	0.03431	0.03963	0.04143
1.8	0.03501	0.06101	0.07887	0.08929	0.09271	0.01334	0.02426	0.03236	0.03735	0.03904
2.0	0.03725	0.06527	0.08472	0.09617	0.09994	0.01262	0.02288	0.03047	0.03513	0.03670
2.5	0.04096	0.07232	0.09444	0.10758	0.11194	0.01115	0.02009	0.02663	0.03062	0.03200
3.0	0.04293	0.07607	0.09959	0.11364	0.11831	0.01021	0.01829	0.02416	0.02771	0.02890
4.0	0.04448	0.07901	0.10363	0.11839	0.12331	0.00934	0.01666	0.02190	0.02506	0.02611
∞	0.04500	0.08000	0.10500	0.11999	0.12499	0.00900	0.01600	0.02100	0.02400	0.02500

Since a unit strip is being used, b will be 12 in., and d is equal to 12 in. minus the cover. Assume a cover of 4.5 in. and, therefore, d will be 7.5 in. So, R_u required will be

$R_u = 227.4$ in-k/[(12 in)/(7.5 in)^2] = 0.337 ksi = 336.9 psi

Now, the balanced reinforcement ratio ρ_b is given by

$$\rho_b = \frac{0.85 f_c'}{f_y}\beta_1\left(\frac{87{,}000}{87{,}000 + f_y}\right) = \frac{(0.85)(3000)}{60{,}000}(0.85)\left(\frac{87{,}000}{87{,}000 + 60{,}000}\right)$$
$$= 0.02138$$

Assume ρ is 0.375 ρ_b, then ρ is equal to 0.00802 and the required ρ is equal to the following:

$$required\ \rho \cong \frac{(req'd\ R_u)(0.375\rho_b)}{R_u}$$

But first, the R_u for this required 0.375 ρ_b condition must be determined:

$R_u = \rho f_y\,(1 - ½\,\rho m) = (0.00802)(60{,}000)[1 - 0.5(0.00802)(14.12)]$
$= 454.0$ psi

Since $m = f_y/(0.85 f_c') = 60{,}000/(0.85)(5{,}000) = 14.12$

Therefore, the required ρ will be

$$required\ \rho \cong \frac{(336.9)(0.00802)}{454.0} = 0.00595$$

From this, the required area of steel, A_s, can be determined using

$$A_s = \rho bd = 0.00595(12)(7.5) = 0.54 \text{ in}^2\text{/ft width}$$

The reinforcing provided in the base of the dam section consists of No. 8 reinforcing bars spaced at 5 in. and results in an average area of steel of 1.90 in^2/ft width. The reinforcing provided is therefore more than adequate for the moment computed using plate theory. A check for cracking may be performed in a manner similar to the one done in Example 1 of Appendix E.

This approach could also be used on side panels for the transportation case. The Timoshenko and Woinowksy-Krieger text has a case for simply supported rectangular plates acted on by a hydrostatic load that could be used on side panels.

Design from In-Plane Loads

Plate theory accounts only for the bending in the panel due to out-of-plane loading and therefore does not account for loadings that may be acting in the plane of the panel. Shell theory can be used to account for in-plane loads, but it does not have the simple solutions that are provided in plate theory.

To account for in-plane loading for the example described in the previous section, the diaphragms located throughout the float-in section can be treated as compression members. Tributary areas for each diaphragm must be determined, and then the load from these areas can be applied to the diaphragms.

As stated previously, the float-in section will have a 10-ft draft during the float-in process. Consider one of the diaphragms in the float-in section that is normal to direction of flow. In general, the spacing of these diaphragms is at 16 ft. Conservatively, the bottom 1 ft of the diaphragm may be evaluated. Since the floor slab is 1 ft thick, the axial load should be based on a water depth of 8.5 ft. So, for a 1-ft-high section, the axial load P acting at the end of a diaphragm would be

$$P = \tfrac{1}{2}\, bhw = \tfrac{1}{2}\,(8.5 \text{ ft} \times 62.5 \text{ lb/ft}^3)(1 \text{ ft})(16 \text{ ft})$$
$$= 4{,}250 \text{ lb}$$

where

b = pressure at midheight of the 1-ft strip
h = height of strip = 1 ft
w = width of tributary area

As before, this computed load must be factored. The factored load is

$$P_u = 1.4(4{,}250 \text{ lb}) = 5{,}950 \text{ lb}$$

So now the design axial load strength P_n must be computed and compared against the above value for P_u. This capacity can be determined by using either Equation 4-2 or 4-12 of EM 1110-2-2104 (HQDA 1992) and ignoring the compressive strength of any of the reinforcing steel. The resulting equation is

$$\phi P_n = 0.8\phi\,(0.85 f_c A_g)$$

where

ϕ = strength reduction factor

A_g = gross area of section

From Section 9.3 of ACI 318, ϕ for an axial member in compression is 0.7. Therefore, ϕP_n will be

$$\phi P_n = 0.8(0.7)(0.85)(3{,}000 \text{ psi})(12 \text{ in} \times 12 \text{ in})$$
$$= 205{,}632 \text{ lb}$$

So,

$$\phi P_n = 205{,}632 \text{ lb} >> P_u = 5{,}950 \text{ lb}$$

The diaphragm is therefore well within the limits permitted. This same approach could likewise be used for looking at compression of diaphragms that are longitudinal to the direction of flow, as well as on the base slab of the section.

Computer Applications

As discussed in Chapter 1 of this report, there are a number of programs that could be used to assist the structural engineer in performing the design of float-in or lift-in units. This portion of Appendix F will discuss in more detail the possible analyses that could be performed.

Grid models

Grid models were used in the design of the Braddock Dam float-in sections, as discussed briefly in Appendix E. Grid models separate a structure into a series of one-dimensional (1-D) beam members. These members are then given properties associated with the manner in which the structure was divided. In the case of the example in Appendix E, diaphragms from the float-in section were treated as webs, and the slab and top panel of the section were treated as flanges. This required determining the effective widths of the flanges so that an area and moment of inertia could be calculated.

Grid models such as the one used during design of the Braddock Dam are effective in determining the overall behavior of a structural system. They also allow the application of loads in three dimensions, which is critical for float-in and lift-in structures. Another advantage of a grid model is that it provides output that can be used directly for design calculations. Output will include a moment, shear force, and axial force.

The primary drawback of using a grid model is that it does not provide information about localized stresses. For instance, for the Braddock Dam example, some additional analysis beyond the grid model analysis would need to be performed to ensure that the connection of the top and bottom panels to the diaphragm is adequate. There may be other locations where more detailed analysis is needed to ensure that localized failure of the section does not occur. Therefore, if a grid model is used, care should be taken in evaluating areas that may require a localized design.

Model using shell elements

Shell theory was briefly mentioned above as a possibility for analyzing thin-wall panels, but it was concluded that analyzing structures by hand using this method was ineffective due to its complexity. However, some programs contain shell elements that could be used to analyze thin-wall panels.

Shell elements use the theory of shells for their mathematical formulation. Shell elements are 2-D elements that can be loaded both in-plane and out-of-plane, and will provide results needed for design, including shear, thrust, and bending moments. Shell elements are laid out in a 2-D grid for each panel modeled, and a thickness is specified for each element.

The shell element is a powerful element and may not be available in all PC-based software packages. Use of shell elements to evaluate a problem may also require a powerful PC due to the increased number of degrees of freedom that will be required. Despite these drawbacks, use of shell elements is a viable alternative in evaluating thin-wall panels because of the versatility and capability this method offers for providing the desired output. Shell elements will also provide a certain degree of information regarding localized stresses that cannot be obtained from grid models.

Three-dimensional brick models

Three-dimensional finite element brick models are another option for evaluating thin-wall panels. Since 3-D brick models can be used to model the actual geometry of the structure, this type of modeling would likely give the most accurate information about localized stresses. However, it is a computationally exhaustive. It is unlikely that an entire float-in or lift-in section modeled with brick elements could be analyzed on a PC. Another shortcoming of using brick elements is the tremendous amount of data that are generated from the model, making it difficult to properly evaluate the results. Finally, results from brick elements are provided in the form of stresses acting in the direction of the coordinate axes. This output cannot be used directly in design calculations. Stress distributions at given sections must be plotted to obtain the shear, thrusts, and moments using this method.

While using brick elements for an entire section may not be practical, or even possible, using brick elements to investigate a localized area is much more feasible. By investigating a local area, the number of elements can be reduced to a number that is acceptable computationally as well as from the standpoint of data management. Of course, when a portion of a structure is analyzed separately

from the rest of the structure, selection of the boundary conditions for the portion of the structure being analyzed is critical. For something like a 3-D model, it may be wise to include the performance of two analyses with different boundary conditions to determine the effects of the chosen boundary conditions on the solutions obtained.

Summary

Other methods can be used in the analysis of thin-wall panels; however, the methods described above give an indication of the possibilities that exist. These methods can be used to perform the designs of thin-wall panels, or the designer may wish to use them as a check against other designs.

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14. ABSTRACT

This report addresses many of the design issues that designers will face with respect to thin-wall precast concrete panels as the U.S. Army Corps of Engineers continues with its innovative approach to construction of navigation structures using lift-in, float-in, and in-the-wet construction methods.

The report focuses on the issues of concrete panel design that relate to the special considerations associated with the innovative methods currently being used by the Corps. This report is intended to supplement guidance criteria currently available in Engineer Manuals and Engineer Technical Letters, as well as the industry standards such as the American Concrete Institute's building code and the American Association of State Highway and Transportation Officials' Bridge Code.

Special consideration is given to the applicability of thin-wall precast panels, the materials used in their construction, and constructibility issues associated with them. Other topics that are addressed include loads to consider during precast operations, considerations with regard to connection of precast items, issues associated with in-fill concrete, and requirements and guidelines regarding construction details. Several chapters of the report are devoted to issues that are directly related to design, such as serviceability requirements, strength requirements, flexural design, shear and torsion design, and fatigue strength design. Finally, information is presented on design with respect to composite construction and loading combinations that should be considered for thin-wall panels.

15. SUBJECT TERMS

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